NATIONAL WEATHER SERVICE
OPERATIONAL DYNAMIC WAVE MODEL

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INTRODUCTION

The National Weather Service (NWS) hydrology program is to provide accurate and timely hydrologic information to the general public. This includes flood forecasts, as well as day-to-day river forecasts which are used for water supply, navigation, irrigation, power, reservoir operation, recreation, and water quality interests. Twelve River Forecast Centers prepare the forecasts which are disseminated to the public throughout the United States via local Weather Service Forecast Offices.

In the late 1960's, NWS began moving from an index type catchment run-off model to a continuous conceptual hydrologic model with a strong physical basis (Monro and Anderson, 1974). The new conceptual model is now being implemented throughout the United States.

Where runoff generated from precipitation input to the conceptual model aggregates in fairly large, well-defined channels (rivers), it is transmitted downstream by routing techniques of the hydrologic or storage routing variety, e.g., the lag and K technique (Linsley, et al., 1958). Although the hydrologic routing techniques function adequately in many situations, they have serious shortcomings when the unsteady flows are subjected to backwater effects due to reservoirs, tides, or inflows from large tributaries. When channel bottom slopes are quite mild, the flow inertial effects ignored in the hydrologic technique also become important.

In the early 1970's, the NWS Hydrologic Research Laboratory began developing a dynamic wave routing model based on an implicit finite difference solution of the complete one-dimensional St. Venant equations of unsteady flow. This hydrodynamic model, known as DWOPER (Dynamic Wave Operational Model), has recently begun to be implemented where backwater effects and mild bottom slopes are most troublesome for hydrologic routing methods. It is either in operational service or in the process of being implemented on the Mississippi, Ohio, Columbia, Missouri, Arkansas, Red, Atchafalaya, Cumberland, Tennessee, Willamette, Platte, Kansas, Verdigris, Ouachita, and Yazoo Rivers.

DWOPER features the ability to use large time steps for slowly varying floods and to use cross-sections spaced at irregular intervals along the river system. The model is generalized for wide applicability to rivers of varying physical features, such as irregular geometry, variable roughness parameters, lateral inflows, flow diversions, off-channel storage, local

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head losses such as bridge contraction-expansions, lock and dam operations, and wind effects. It is suited for efficient application to dendritic river systems or to channel networks consisting of bifurcations with weir-type flow into the bifurcated channel. DWOPER has a highly efficient automatic calibration feature for determining the optimum roughness coefficients for either a single channel or system of interacting channels. Extensive data management programming features allow the model to be used in a day-to-day operational forecasting environment with minimal card coding required. It is also equally applicable for simulating unsteady flows for purposes of engineering planning, design, or analysis.

MODEL DESCRIPTION

Mathematical Basis - the basis for DWOPER is a finite difference solution of the conservation form of the one-dimensional equations of unsteady flow consisting of conservation of mass and momentum equations, i.e.,

$$\frac{\partial Q}{\partial x} + \frac{\partial (A+A_o)}{\partial t} - q = 0 \qquad (1)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2/A)}{\partial x} + gA\left(\frac{\partial h}{\partial x} + S_f + S_e\right) - qv_x + W_f B = 0 \qquad (2)$$

in which Q is discharge, A is cross-sectional area, A_o is off-channel cross-sectional area wherein flow velocity is considered negligible, q is lateral inflow or outflow, x is distance along the channel, t is time, g is gravity acceleration constant, v_x is the velocity of lateral inflow in the x-direction, W_f is the wind term, B is the channel top width and S_f is the friction slope defined as:

$$S_f = \frac{n^2 |Q| Q}{2.2\ A^2 R^{4/3}} \qquad (3)$$

in which n is the Manning roughness coefficient and R is the hydraulic radius. The term S_e is defined as:

$$S_e = \frac{K_e \partial (Q/A)^2}{2g\ \partial x} \qquad (4)$$

in which K_e is the expansion-contraction coefficient.

Equations (1) and (2) are nonlinear partial differential equations which may be solved without necessary simplifications by finite difference techniques of explicit or implicit variety. Explicit methods, although simpler in application are not suitable for application of the equations to long-term unsteady flow phenomena such as flood waves because they are restricted by mathematical stability considerations to very small computational time steps (on the order of a few minutes); this causes the explicit techniques to be very inefficient in the use of computer time. Implicit finite difference techniques, however, have no restrictions on time step size other than accuracy considerations.

The "weighted four-point" implicit scheme first used by Preissmann (1961) and recently by Quinn and Wylie (1973), Chaudry and Contractor (1973), and Fread (1973a, 1974) appears to be the most advantageous of the various implicit schemes which have been proposed from time to time because it can readily be used with unequal distance steps and its stability-convergence properties can be controlled. In the weighted four-point scheme, the continuous x-t region in which solutions of h and Q are sought is represented by rectangular net of discrete points, as show in Figure 1 at equal or unequal intervals of Δx and Δt along the x and t axes, respectively. Each point is identified by a subscript (i) which designated the x position and a superscript (j) for time position. The time derivatives are approximated by:

$$\frac{\partial K}{\partial t} = (K_i^{j+1} + K_{i+1}^{j+1} - K_i^j - K_{i+1}^j)/2\Delta t \qquad (5)$$

in which K represents any variable. The spatial derivatives are approximated by a finite difference quotient positioned between two adjacent time lines according to weighting factors Θ and $1-\Theta$, i.e.,

$$\frac{\partial K}{\partial x} = \Theta(K_{i+1}^{j+1} - K_i^{j+1})/\Delta x + (1-\Theta)(K_{i+1}^j - K_i^j)/\Delta x \qquad (6)$$

and variables other than derivatives are approximated in a similar manner, i.e.,

$$K = \Theta(K_i^{j+1} + K_{i+1}^{j+1})/2 + (1-\Theta)(K_i^j + K_{i+1}^j)/2 \qquad (7)$$

when Θ equals 1.0, a fully implicit scheme is formed; this was used by Baltzer and Lai (1968) and Amein and Chu (1975). A box scheme such as used by Amein and Fang (1970) results if Θ is fixed at 0.5. The influence of the Θ weighting factor on the stability and convergence properties was examined by Fread (1974), who concluded that the accuracy decreases as Θ departs from 0.5 and approaches 1.0. This effect becomes more pronounced as the time step size increases. DWOPER allows Θ to be an input parameter. A value of 0.55 is often used to minimize loss of accuracy while avoiding weak or pseudo instability as reported by Baltzer and Lai (1968) and Fread (1975) when Θ of 0.5 is used.

Substitution of the finite difference quotients defined by eqs. (5-7) into eqs. (1) and (2) for the derivatives and non-derivative terms produces two algebraic equations which are nonlinear with respect to the unknowns h and Q at the net points on the j+1 time line. All terms associated with the j^{th} time line are known from either the initial conditions or previous computations. The initial conditions are values of h and Q at each computational point (node) along the x-axis for the first time line (j+1). They are obtained from a previous unsteady flow solution, or they can be estimated since small errors in the initial conditions dampen out within a few time steps.

The two nonlinear algebraic equations cannot be solved in a direct (explicit) manner since there are four unknowns, h and Q, at points i and i+1 on the j+1 time line, and only two equations. However, if similar equations are formed for each of the N-1 Δx reaches between the upstream and downstream boundaries, a total of 2N-2 equations with 2N unknowns results. (N denotes the total number of computational points or cross-sections.) Then prescribed boundary conditions, one at the upstream extremity of the river and one at the downstream extremity, provide the necessary two additional equations required for the system to be determinate. The resulting system of 2N nonlinear equations with 2N unknowns is solved by a functional iterative procedure, the Newton-Raphson method (Amein and Fang, 1970).

In the iterative procedure, trial values obtained via linear or parabolic extrapolation from solutions of h and Q at previous time steps are assigned to the 2N unknowns. Substitution of these into the system of 2N nonlinear equations yields a set of 2N residuals. The Newton-Raphson method seeks to reduce the residuals to an acceptable tolerance level which is usually achieved within one or two iterations. If the Newton-Raphson method is applied only once each time step, the solution procedure degenerates to the equivalent of a quasi-linear finite difference approximation of eqs. (1) and (2) similar to that used by Chen and Simons (1975) and others. The linear difference solution requires smaller time steps than the nonlinear formulation for the same degree of accuracy. However, for very gradually varying flows, the time step size which may be fixed for reasons of data availability and user requirements can be such that the loss of accuracy using the linear formulation or its equivalent is insignificant.

In the Newton-Raphson method, a system of 2N x 2N linear equations are generated. The coefficient matrix of the system is composed of partial derivatives which are functions of the unknowns; however, the elements in the coefficient matrix can be assigned numerical values by substituting the trial values of unknowns. The coefficient matrix is related to the set of 2N residuals by a set of 2N corrections to the original trial values of the unknowns. It is the 2N corrections that are sought in the solution of the 2N x 2N linear system. The coefficient matrix has a banded structure with at most four elements in any row. This property allows the use of a special modified Gaussian elimination algorithm for solving the system (Fread, 1971). Modification of the elimination algorithm reduces the core storage from $4N^2$ to 8N and the number of computational operations are reduced from the order of $(8/3N^3 + 4N^2)$ to 38N. The increase in computational efficiency is critical to the feasibility of the implicit solution technique.

Initial Conditions - DWOPER allows initial conditions to be obtained from the following sources:

1) Estimated stages and discharges at each cross-section are read in;
2) Observed stages at each cross-section where a river gage is located are read in; stages at intermediate cross-sections are linearly interpolated within the model; observed discharge at the upstream extremity of the main stem river and each tributary are also read in; all downstream discharges are determined by summation of flows from the upstream to downstream boundaries including tributary inflow to the main stem and lateral inflow occurring along either the main stem or tributaries;
3) Computed stages and discharges which have been saved from a previous unsteady flow simulation; and
(4) Assumed steady flow to obtain discharges and backwater computation to obtain stages.

In each case, the unsteady flow equations are solved for several time steps using the initial conditions together with boundary conditions which are held constant during the time steps. This allows the errors in the initial conditions to dampen out which results in the initial conditions being more nearly error free when the actual simulation commences and transient boundary conditions are used.

Boundary Conditions - Boundary conditions must be specified in order to obtain solutions to the St. Venant equations. In fact, in most unsteady flow problems, the unsteady disturbance is introduced into the flow at the boundaries or extremities of the river system. DWOPER can readily accommodate either of the following boundary conditions at the upstream extremities of the river system:

1) known stage (water surface elevation) hydrograph, $h_1(t)$; or
2) known discharge hydrograph, $Q_1(t)$.

Downstream boundary conditions included as options in DWOPER are:

1) known stage hydrograph, $h_N(t)$;
2) known discharge hydrograph, $Q_N(t)$; or
3) a known relationship between stage and discharge such as a rating curve.

With respect to the rating curve boundary condition, the rating may be single-valued and read in as tabular (piece-wise linear) values of stage and discharge with linear interpolation provided internally for intermediate values. The rating may also be a loop rating curve generated internally from cross-section and roughness properties of the downstream extremity and the instantaneous water surface slope at the previous time step. The rating may also be a weir-type relation of the form $Q_N = C_1 \ (h_N^{J+1} - H_w)^{C2}$ in which the weir coefficients (C_1 and C_2) and the weir crest elevation (h_w) are read in.

Hydrographs of stage or discharge may be read in at a constant time interval which is also an input to the model or they may be read in at irregular intervals of time, in which case, the interval is also specified each time it changes.

Cross-Sections - Irregular as well as regular geometrical shaped cross-sections are acceptable in DWOPER. Each cross-section is read in as tabular values of channel width and elevation, which together constitute a piece-wise linear relationship. Experience has shown that in almost all instances the cross-section may be sufficiently described with eight or less sets of widths and associated elevations. A low-flow cross-sectional area which can be zero is used to describe the cross-section below the minimum elevation read in. From this input, the cross-sectional area associated with each of the widths is initially computed within the model. During the solution of the unsteady flow equations, any areas or widths associated with a particular water surface elevation are linearly interpolated from the piece-wise linear relationships of width to elevation read in or the area-elevation sets initially generated within the model.

Cross-sections at gaging station locations are always used as computational points in the x-t plane. Cross-sections are also specified at points along the river where significant cross-sectional changes occur or at points where major tributaries enter. Typically, cross-sections for large rivers with slowly varying transients may be spaced as much as 5-20 miles apart. The maximum distance between cross-sections is dependent on the channel properties, duration and rate of rise of the wave, and the computational step size of Δx and Δt. The relation of each to the accuracy of the simulation is defined by Fread (1974). Usually, "average" cross-sections are placed midway between cross-sections associated with gaging stations or significant geometry changes. The average cross-section is a weighted average of the cross-sectional properties of the intervening reach. The average cross-section is obtained from a special data processing program in which cross-sections at all sections such as crossings, bends, or other changes in cross-section geometry which violate the assumption of linear variation between adjacent cross-sections are averaged together to obtain a weighted average. This distance between cross-section is used as the weighting factor.

Off-Channel Storage - Dead storage areas wherein the flow velocity in the x-direction is considered negligible relative to the velocity in the active area of the cross-section is a feature of DWOPER. Such dead or off-channel storage areas can be used to effectively account for embayments, ravines, or tributaries which connect to the flow channel but do not pass flow and serve only to store the flow. Another effective use of off-channel storage is to model a heavily wooded flood plain which stores a portion of the flood waters passing through the channel. In each of these cases, the use of zero velocity for the portion of the flood waters contained in the dead storage areas results in a more realistic simulation of the actual flow than using an average velocity derived from the main flow channel and the dead storage area. The off-channel storage cross-sectional properties are described in the same way as the active cross-sectional areas, i.e., for each section, a table of top widths and elevations is read in along with the area associated with the lowest elevation. A table of area-elevation is created within DWOPER and intermediate storage top widths or areas are linearly interpolated from the two tables as required.

Roughness Coefficients - Manning's n is used to describe the resistance to flow due to channel roughness caused by bed forms, bank vegetation and obstructions, bend effects, and eddy losses. The Manning n is defined for each channel reach bounded by gaging stations and is specified as a function of either stage or discharge according to a piece-wise linear relation with both n and the independent variable (h or Q) read in to DWOPER in tabular form. Linear interpolation is used to obtain n for values of h or Q intermediate to the tabular values.

Simulation results are often very sensitive to the Manning n. Although in the absence of necessary data (observed stages and discharges), n can be estimated; however, best results are obtained when n is adjusted to reproduce historical observations of stage and discharge. The adjustment process is referred to as calibration. This may be either a trial-error process or an automatic iterative procedure available within DWOPER. The automatic calibration feature is described later.

Lateral Inflows - DWOPER incorporates tributary inflows via the lateral inflow term, q, in eqs. (1) and (2). The inflows are considered to be independent of flows occurring in the river to which they are added. They are read in as a time series of flows with constant or variable time intervals. They may be specified for any Δx reach along the river. The flow is specified in cfs; it is the sum of all lateral inflow occurring within a particular Δx reach. Outflows may also be simulated by assigning a negative sign to the flow value in the time series. Linear interpolation is used to provide flow values at times other than those of the time series which are determined by the time intervals associated with the flows.

Local Losses - The effects of local head losses incurred at severe contractions and/or expansions such as bridge openings are accounted for by the term S_e as defined by eq. (4). This necessitates an expansion-contraction coefficient associated with each Δx reach to be read in. The local head loss is in addition to the head loses incurred by the flow resistance due to channel roughness associated with the Manning n.

Wind Effects - The effect of wind resistance on the surface of the flow is accounted for by the term W_f in eq. (2). W_f is defined as:

$$W_f = C_w \, (V_r \cos \omega)^2 \tag{8}$$

in which V_r is the velocity of the wind relative to the velocity of the channel flow, ω is the angle between the wind direction and channel flow direction, and C_w is the nondimensional wind coefficient. C_w may be estimated from empirical studies or it may be assigned a value via a trial-error calibration process.

Lock and Dam Condition - A river system may include small dams with gates to pass the river flow in such a way as to maintain certain water surface elevations on the upstream side of the dam. Usually associated with the dam is a lock for allowing navigation of river craft and barges past the dam. DWOPER can accommodate any number of lock and dam installations within the river system being simulated. A "through" computation scheme is used as opposed to separating the river system into discrete portions because of the lock and dam and specifying external boundary conditions applicable to the lock and dam. The through computation scheme allows the simultaneous simulation of the entire river system including portions with lock and dams. This facilitates data preparation and allows a correct simulation of backwater effects when the tailwater elevation below that dam raises to an elevation such that the pool elevation on the upstream side is no longer controlled by operation of the gates.

In the through computation scheme a specified critical tailwater elevation which is read in for a particular lock and dam is used to determine if the pool elevation is controlled by the gate operation or by channel flow. If the simulated tailwater is less than the critical elevation, the conservation of mass equation, eq. (1), is replaced by

$$Q_i^{j+1} - Q_{i+1}^{j+1} = 0 \qquad (9)$$

and the conservation of momentum equation, eq. (2), is replaced by

$$h_i^{j+1} - h_t = 0 \qquad (10$$

where h_t is the target pool elevation which the dam tender attempts to maintain via operation of the gates. The target pool elevation may be a constant value or it may be specified as a function of time and read in as a time series.

When the simulated tailwater elevation exceeds the critical tailwater elevation, the flow passes through the dam according to eqs. (1) and (2) as in any other Δx reach.

Flow Diversions - Special flow diversions through diversion control structures are accommodated within DWOPER. The model simulates the amount of flow diverted on the basis of the current simulated river elevation on the headwater side of the diversion structure, the type and number of diversion gates in operation, and the tailwater elevation on the downstream side of the diversion structure. The latter two must be specified and read in as input data. Currently, DWOPER is programmed for the special flow diversion structures known as Old River Diversion, Morganza Diversion, and Bonnet Carre Diversion; these are all used for diverting flows in the lower Mississippi River. As other diversion structures are encountered in other river systems, they will be added to DWOPER as special subroutines.

Dendritic River Systems - Although the implicit formulation of the unsteady flow equations is well suited for simulating unsteady flows in a system of rivers -- in that the response of the system as a whole is determined during each time step -- particular care must be given to maintain the necessary solution efficiency as mentioned previously with regards to the matrix solution technique of the Newton-Raphson procedure. An efficient solution technique for dendritic (tree-type) river systems is utilized in DWOPER. This technique, as described by Fread (1973), solves during a time step the unsteady flow equations first for the main stem and then for each tributary of the river system. The tributary flow at the confluence of the tributary and main-stem river is treated as lateral flow q which is first estimated when solving the equations first for the main stem.

The tributary flow depends on its upstream boundary condition, lateral inflows along its reach, and the water surface elevation at the confluence which is obtained during the simulation of the main stem. Due to the interdependence of the flows in the main stem and its tributaries, an iterative or relaxation procedure is necessary. Convergence is attained when the estimated tributary flow at the confluence is sufficiently close to the computed flow for the tributary at its downstream boundary using the main stem water surface elevation for the downstream boundary condition. Usually, one or two iterations is sufficient for convergence to a suitable tolerance.

DWOPER can accommodate any number of tributaries. Although the iterative algorithm is designed for first order tributaries, systems with second order tributaries may sometimes be accommodated by reordering the system, i.e., selecting another branch of the system as the main stem.

Weir-Flow Bifurcations - In DWOPER, any number of Δx reaches along a channel may bypass flow to another channel which connects back into the former channel at some point downstream from the bifurcation. The flow in the bypass channel which may affect the weir flow is accounted for by a submergence correction to the weir flow. The crest elevation of the overbank section which acts as the weir-flow bypass is specified. Each section has a discharge coefficient which may be estimated or obtained through trial-error calibration. The location along the channel where the bifurcation(s) occur, the average crest elevation of each such Δx reach, and the discharge coefficient are read in as input data.

Automatic Calibration - A critical task in the application of one-dimensional hydrodynamic models in natural rivers in dendritic systems is the determination of the roughness parameter in the friction slope term of the momentum equation. The roughness parameter often varies with discharge or stage and with distance along the river. The DWOPER model contains an internal feature which can be accessed to automatically determine the optimum roughness parameter which will minimize the difference between computed and observed values. The automatic calibration feature is a simple and highly efficient optimization technique (Fread and Smith, 1978), for determining the continuous piece-wise linear variation of the roughness parameter with discharge (or stage) for each reach of the river bounded by gaging stations. The optimization technique is based on a decomposition principle which simplifies the treatment of complex river systems of dendritic configuration. Computational requirements are less than twice that required for a normal simulation run in which the roughness parameters are known and calibration is not required.

In the automatic calibration procedure, each reach between gaging stations is calibrated (i.e., a set of optimum roughness coefficients is obtained) sequentially, commencing with the most upstream reach and progressing reach by reach in the downstream direction. Tributaries are calibrated before the main-stem river and their flows are added to the main stem as lateral inflows. Discharge is input at the upstream boundary of each river, while observed stages at the downstream gaging station of each reach is used as the downstream boundary condition. Computed stages at the upstream boundary are tested against observed stages at that point. Statistics of bias and root-mean-square (RMS) error are computed for several ranges of discharge or stage so that the roughness coefficient can be calibrated as a function of discharge (stage). For each range of discharge, the adjustment uses the bias to determine the magnitude and whether the change in roughness should be positive or negative. Adjustments are automatically made to the roughness coefficients for the reach and the one reach system is rerun. The cycle is repeated until a minimum RMS error for the reach is found.

The discharges computed at the downstream boundary using the optimum roughness coefficients associated with the minimum RMS error are stored internally and then input as the upstream boundary condition for the next lower reach. An optimum set of roughness coefficients which minimizes RMS errors throughout the river system is determined by proceeding through the river system one reach at a time in a sequence from upstream to downstream and from tributaries to main stem.

Data Management and Display Features - DWOPER consists of three functional elements, each consisting of several subroutines. One element contains the basic algorithms associated with solving the unsteady flow equations for simulating unsteady flows in a river system. The second element is a data management module which provides channel configuration, channel properties such as cross-section geometry and roughness, boundary conditions, and initial conditions for the basic dynamic wave computational element. The third element displays the simulation results in either graphical or tabular form.

Preparation of the data for simulation of a river system requires a substantial amount of work. The river system configuration, cross-sections, etc. must be determined and coded. Stage and discharge data for the boundary and initial conditions must be determined and card coded. This initial work cannot be avoided; however, the data management module does substantially reduce the time and effort required to use the model on a day-to-day operational basis. The data initially card coded to simulate a particular river system are kept on disk, a direct access peripheral mass storage, and only the updated information for boundary conditions need be card coded and read in before a new simulation can be made.

The data stored on disk are of two types: stationary data which does not change with time and stage-discharge data which must be updated as new observations are reported. The stationary data are stored in "carryover" files and stage-discharge data are stored in "hydrograph" files. In order to perform a forecasting run, the river system configuration and physical properties must be determined by retrieving the data in a carryover file and the stage-discharge data must be retrieved from a hydrograph file. After the initial simulation run, the initial conditions which consist of the stages and discharges at every computational point in the river system are available in the carryover file as computed stages and discharges which have been stored from a previous run.

Each carryover and hydrograph file is identified by a unique name. Since any number of carryover and hydrograph files can exist in mass storage, DWOPER searches the carryover files for the name that matches that specified for the current simulation run. With the correct carryover file retrieved, DWOPER then searches the hydrograph files for those hydrographs labeled with the same names as specified in the carryover file. The hydrographs in the file have names which must match the labels in the carryover file. When a match is found, data for the appropriate time period are extracted from the hydrograph file for use in the simulation. The label system allows the two types of data (stationary and hydrograph) to be stored separately and combined properly prior to the computation sequence of a simulation run. Thus, new data which arrive in day-to-day operations need to be added to the hydrograph file.

The data management element, the dynamic wave computational element, and the forecast display element are accessed by commands. Each command causes the program to branch to appropriate subroutines where the desired function is performed. Some of the more significant data management commands are:

1) COINIT - The Carryover file is initialized, i.e., the river configuration, cross-section properties, etc., are read in.
2) HINIT - The Hydrograph file is initialized, i.e., the hydrograph values are read in.
3) COEDIT - Any stationary data contained in the carryover file may be updated by simple reference indicators punched on input cards along with the updated value(s).
4) HEDIT - Any data contained in the hydrograph file may be deleted, replaced, or added to by simple label and time period indicators punched on input cards along with the new hydrograph values.
5) COLIST - List the contents of a particular carryover file.
6) HLIST - List the contents of a particular hydrograph file.

Some of the commands used to active the dynamic wave computational element are:

1) RUN - Simulates a river system using data from carryover and hydrograph files stored on disk.
2) ICSAVE - A command used concurrently with RUN command. ICSAVE is used to save the water surface elevations and discharges at all computational points at a specified time. These values are retained in peripheral storage for use in subsequent simulation runs as the appropriate and necessary initial conditions.

3) ALONE - Simulates a river system using data (river system configuration, cross-section properties, hydrograph values at boundaries, etc.) read in on cards at the same time as the ALONE command is read in.

One of the commands used to activate the forecast display element is:

1) DISPLAY - A command used concurrently with RUN command. DISPLAY has various options. One option is: the computed stages, discharges, velocities, and observed stages for all gaging stations within the river system being simulated will be printed for a single time step. Another option is: the computed stages and observed stages at a single gaging station can be displayed for a selected period up to 18 time steps.

If the DISPLAY command is not used, DWOPER will provide the following output:

1) graphs showing computed and observed values of stage or discharge at selected gaging stations;
2) a list of the stages or discharges which are plotted and;
3) bias and root-mean-square error statistics for computed and observed values at each selected gaging station.

Simulation runs using data from carryover and hydrograph files require essentially the same CPU time as a simulation run using the ALONE command in which the data is read in as cards during the initial stages of the simulation.

Computer Core and Computational Requirements - DWOPER has been created using the programming feature "variable dimensioning." This enables the size of the arrays of subscripted variables such as observed hydrograph stages, cross-section top widths, computed stages and discharges, etc., to be changed from one simulation run to the next. There is a maximum total size for the sum of all arrays, but within that bound the allocation of core space among the variables is flexible and is specified as a data input which is retained in the carryover file. This flexibility allows maximum use of the core set apart for DWOPER. Actually, three sizes of DWOPER are used in operational forecasting with the smallest version used on all river systems which will fit within its array size. The small version of DWOPER requires 170 K words having 8 bits per word. The medium and large versions require 235 and 300 K words of memory storage, respectively.

Another programming feature known as "overlaying," in which groups of subroutines are loaded into core as needed, is used to reduce the required core to that mentioned above.

The implicit formulation of the basic dynamic wave computational element allows the time step size to be selected according to accuracy requirements rather than numerical stability considerations. This factor makes DWOPER very efficient in the use of computer time. Efficiency due to the implicit formulation compared to computational requirements of explicit finite difference models is greatest for slowly varying transients in large rivers and decreases as the transient being modeled becomes more rapidly varying. Computational requirements are approximately 0.004 sec/time step/distance step. Some examples of total computational time (IBM 360/195) are:

1) Mississippi-Ohio-Cumberland-Tennessee River system: consisting of 393 miles of river, 45 computational points, 3 months of simulation time, 20 observed and computed hydrographs; slowly varying flood wave simulation using 24-hour time steps required 15 seconds of CPU time.
2) Lower Mississippi River: consisting of 292 miles of river, 25 computational points, 2 months of simulation time, 7 observed and computed hydrographs; slowly varying flood wave simulation using 24-hour time steps required 6 seconds of CPU time.
3) Lower Columbia and Willamette River system: consisting of 155 miles or river, 25 computational points, 9 days of simulation time, 10 observed and computed hydrographs; tidal flows with 12-hour period and 1-hour time steps required 19 seconds of CPU time.

SELECTED APPLICATIONS

Lower Mississippi - DWOPER was applied to a 291.7 mi reach of the Lower Mississippi River from Red River Landing to Venice shown schematically in fig. 2. Six intermediate gaging stations at Baton Rouge, Donaldsonville, Reserve, Carrollton, Chalmette, and Pt. a la Hache were used to evaluate the simulations.

This reach of the Lower Mississippi is contained within levees for most of its length, although some overbank flow occurs along portions of the upper 70 mi. Throughout this reach, the alluvial river meanders between deep bends and relatively shallow crossings; the sinuosity coefficient is 1.6. The low flow depth varies from a minimum of 30 feet at some crossings to a maximum depth of almost 200 feet in some bends. The average channel width is approximately one-half mi. The average channel bottom slope is a very mild 0.034 ft/mi. The discharge varies from low flows of about 100,000 to flood discharges of over 1,200,000 cfs. A total of 25 cross-sections located at unequal intervals ranging from 5-20 miles were used to describe the 291.7 mi reach.

The reach was first automatically calibrated by DWOPER for the 1969 spring flood. Time steps of 24 hours were used. Then, using the calibrated set of Manning n vs. discharge for each reach bounded by gaging stations, the 1969 flood was simulated using stage hydrographs for upstream and downstream boundaries at Red River Landing and Venice, respectively. The simulates stage hydrographs at the six intermediate gaging stations are compared with the observed in figs. 3-5. The RMS error was used as a statistical measure of the accuracy of the calibration. The RMS error varied from 0.17-0.36 ft with an average value of 0.25 ft.

Several historical floods from the period 1959-1971 were then simulated using the calibrated Manning n values obtained from the 1969 flood. An example of simulated vs. observed stages is shown in figs. 6-8 for the 1966 flood. Average RMS errors for all six stations for each of the simulated floods are shown in table 1. The average RMS error for all the floods was 0.47 ft. This compares with 0.25 ft for the calibrated flood of 1969, indicating that for this reach of the Mississippi there is not a significant change in the channel roughness from one flood event to another.

Mississippi-Ohio-Cumberland-Tennessee System - A dendritic river system consisting of 393 miles of the Mississippi-Ohio-Cumberland-Tennessee (MOCT) River system was also simulated using DWOPER. A schematic of the river system is shown in fig. 9. Eleven intermediate gaging stations located at Fords Ferry, Golconda, Paducah, Metropolis, Grand Chain, Cairo, New Madrid, Red Rock, Grand Tower, Cape Girardeau, and Price Landing were used to evaluate the simulation.

In applying DWOPER to this system, the main-stem river is considered to be the Ohio-Lower Mississippi segment with the Cumberland, Tennessee, and Upper Mississippi considered as first-order tributaries. The channel bottom slope is mild, varying from about 0.25 to 0.50 ft/mi. Each branch of the river system is influenced by backwater from downstream branches. Total discharge through the system varies from low flows of approximately 120,000 cfs to flood flows of 1,7000,000 cfs. A total of 45 cross-sections located at unequal intervals ranging from 0.5-21 miles were used to describe the MOCT river system.

The MOCT system was calibrated to determine the n-Q relationship for each of 15 reaches bounded by gaging stations. Time steps of 24 hours were used. About 25 seconds of IBM 360/195 CPU time were required by DWOPER to perform the calibration. The flood of 1970 was used in the automatic calibration process. The average RMS error for all 15 reaches was 0.60 ft. A typical comparison of observed and simulates stages for the Cairo and Cape Girardeau gaging stations is shown in fig. 10.

Using the calibrated n-Q relations, the 1969 flood was simulated with DWOPER. Stage hydrographs at Shawneetown and Chester and discharge hydrographs at Barkley Dam and Kentucky Dam were used as upstream boundary conditions, and a rating curve was used as the downstream boundary condition at Caruthersville. Figure 11 shows the simulated vs. observed stages for Cairo and Cape Girardeau. The average RMS error for the 11 intermediate gaging stations was 0.56 ft.

Columbia-Willamette System - DWOPER was applied to the 130 mi reach of the lower Columbia River below Bonneville Dam, including the 25 mi tributary reach of the lower Willamette River. A schematic of the river system is shown in fig. 12.

This reach of the Columbia has a very flat slope (0.06 ft/mi) and the flows are affected by tides from the Pacific Ocean. The tidal effect extends as far upstream as the tailwater of Bonneville Dam during periods of low flow. Reversals in discharge during low flow are possible as far upstream as Vancouver. A total of 25 cross-sections located at unequal distance intervals ranging from 0.5-12 miles were used to describe the river system. One hour time steps were used in the simulations.

The system was first calibrated for a 4-day period in August 1973. Seven intermediate gaging stations at Warrendale, Washougal, Vancouver, Portland, Columbia City, Rainier, and Wauna were used along with the gaging stations at the extremities of the system, i.e., Bonneville, Oregon Falls, and Tongue Point. Another 5-day period in August 1973 was then simulated using DWOPER and the calibrated n-Q relations. Upstream and downstream boundaries were observed discharges and stages, respectively. The average RMS error for all stations in the simulation run was 0.21 ft. Some examples of simulated and observed stage hydrographs for Warrendale, Vancouver, Portland, and Wauna are shown in figs. 13 and 14.

SUMMARY AND CONCLUDING REMARKS

An operational hydrodynamic model (DWOPER) developed by the Hydrologic Research Laboratory of the National Weather Service is being placed in operational use by River Forecast Centers on a number of major river systems where storage routing methods are inadequate due to the effects of backwater, tides, and mild channel bottom slopes. The model is based on the complete one-dimensional St. Venant equations and belongs to the category of dynamic wave flood routing models. A weighted four-point nonlinear implicit finite difference scheme is used to obtain solutions to the St. Venant equations via a Newton-Raphson iterative technique.

DWOPER has a number of features which make it applicable to a variety of natural river systems for real-time forecasting. It is designed to accommodate various boundary conditions and irregular cross-sections located at unequal distances along a single multiple-reach river or several such rivers having a dendritic configuration. It allows for roughness parameters to vary with location and stage or discharge. Temporally varying lateral inflows, wind effects, bridge effects, off-channel storage, weir-flow channel bifurcations are included among it features. Time steps are chosen solely on the basis of desired accuracy since the implicit finite difference technique is not restricted to the very small time steps of explicit techniques due to numerical stability considerations. This enables DWOPER to be very efficient as to computational time for simulating slowly varying floods of several days duration. An efficient automatic calibration procedure for determining optimum Manning n - stage or discharge relationships from observed data is provided as an option in DWOPER. Data handling requirements for day-to-day river forecasting are minimal due to extensive data management features utilizing disk or tape storage. Operationally, card coding is only required to update hydrograph files with the most recent observations.

Applications of DWOPER to several large river systems have demonstrated its operational efficiency, accuracy, and utility. The model is currently being extended to account for effects of channel sinuosity and flood plains (Fread, 1976), sediment transport (Chen and Simons, 1975), and bank storage on unsteady flows in alluvial rivers. Also, it has been coupled to an unsteady temperature transport model (Bowles, et al., 1977) for development as a river temperature forecasting model.

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Table 1.--Summary of flood simulations in Lower Mississippi River (Red River Landing to Venice) for the years 1959-71

Year	Average RMS error (ft.)	Peak Discharge (1,000 cfs)
1959	0.62	750
1960	.31	850
1961	.47	1,220
1962	.61	1,155
1963	.38	950
1964	.51	1,140
1965	.44	1,040
1966	.38	1,090
1967	.38	700
1968	.36	980
1969*	.25	1,065
1970	.91	1,080
1971	.46	940

*Calibrated

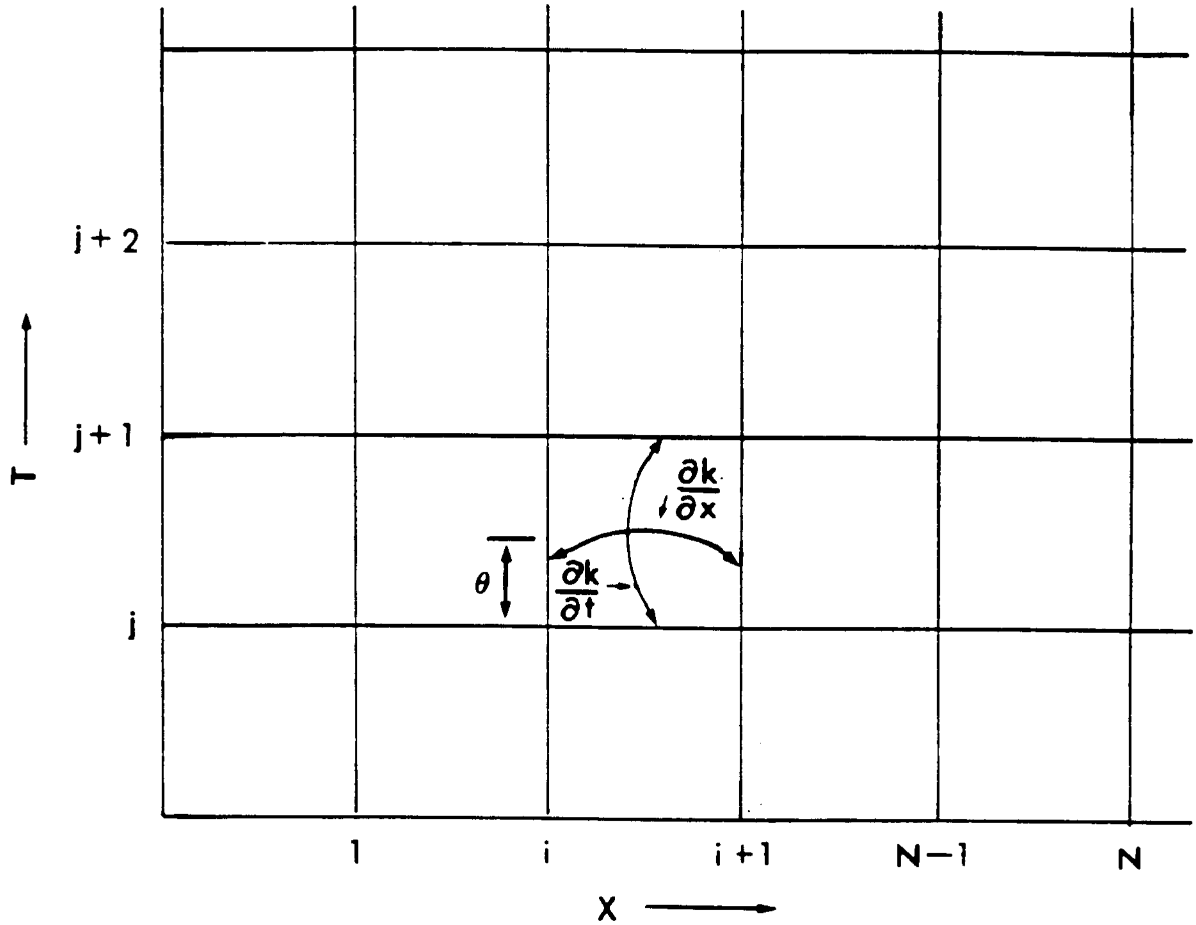


FIG. 1 - DISCRETE X-T SOLUTION DOMAIN

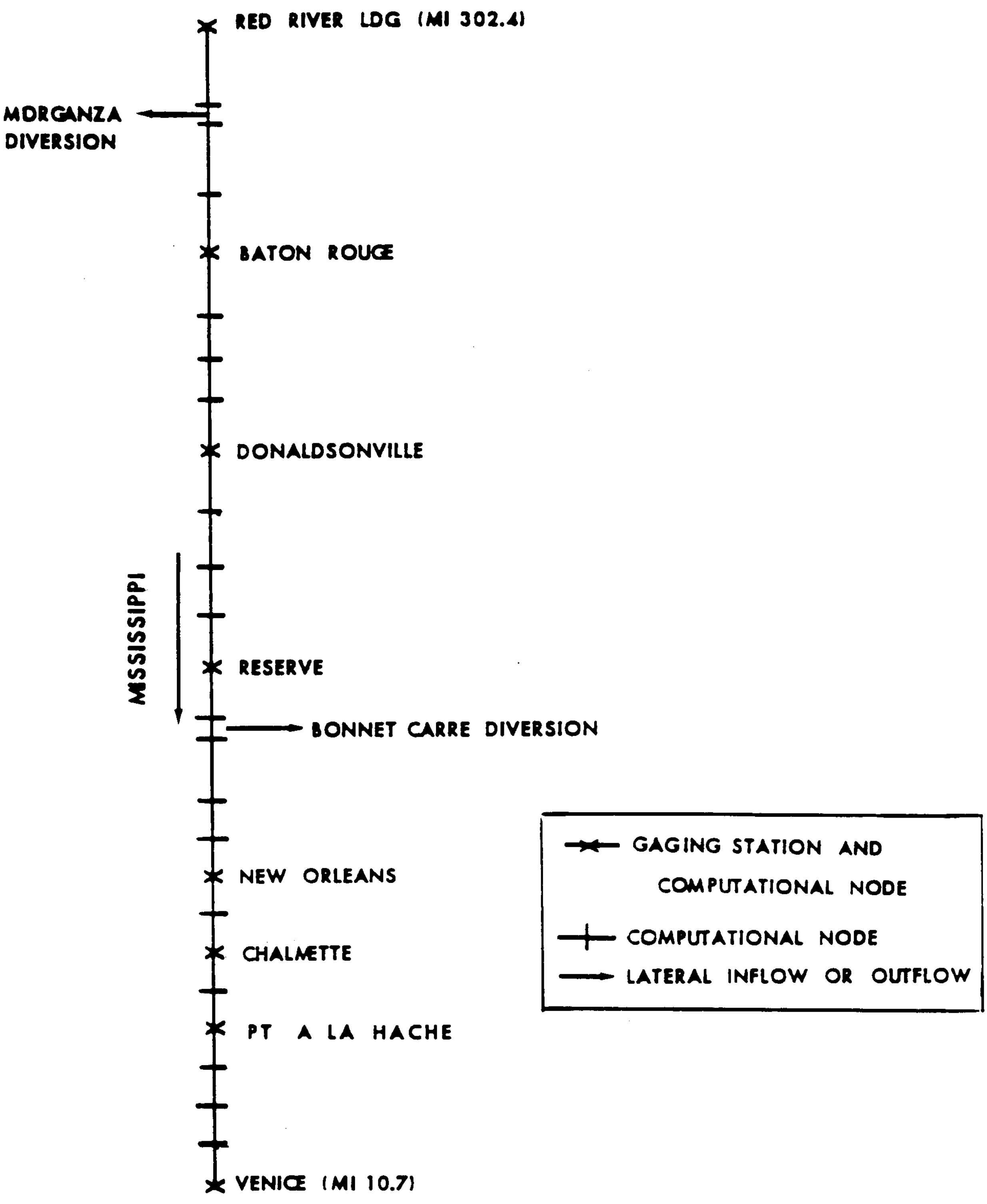


FIG. 2 - SCHEMATIC OF LOWER MISSISSIPPI RIVER

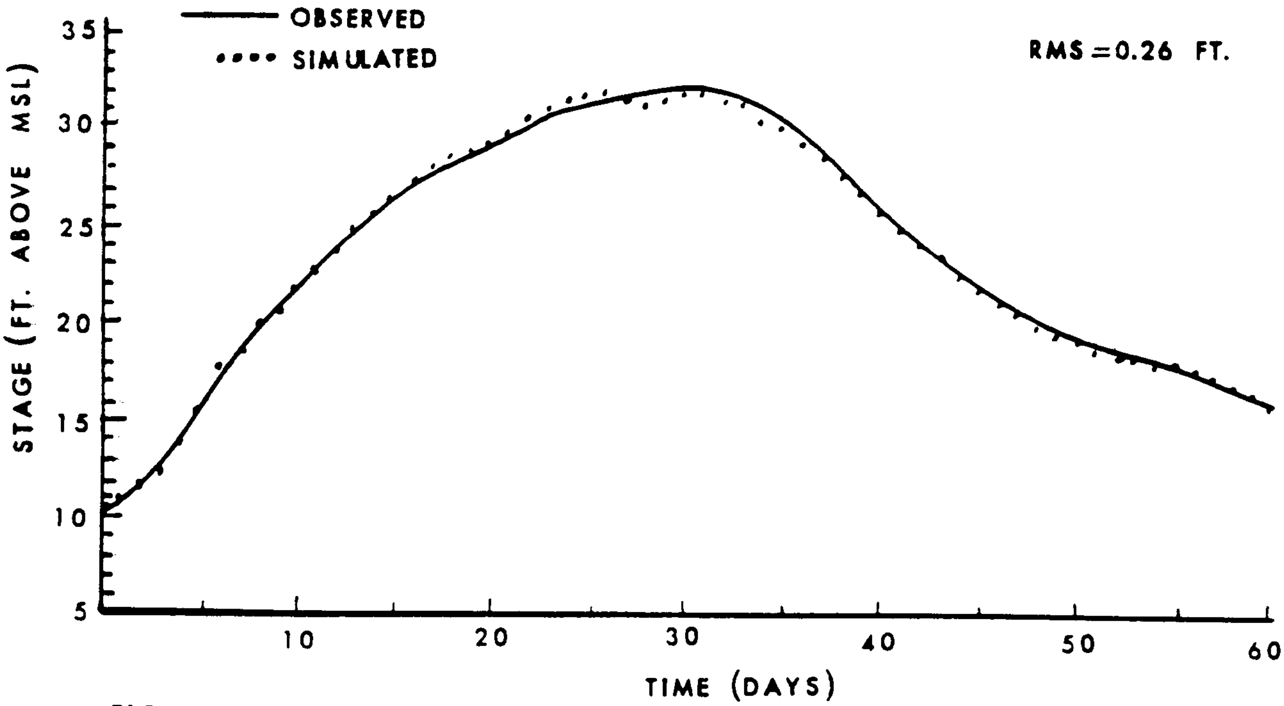


FIG. 3a - OBSERVED VS. SIMULATED STAGES AT BATON ROUGE FOR 1969 FLOOD

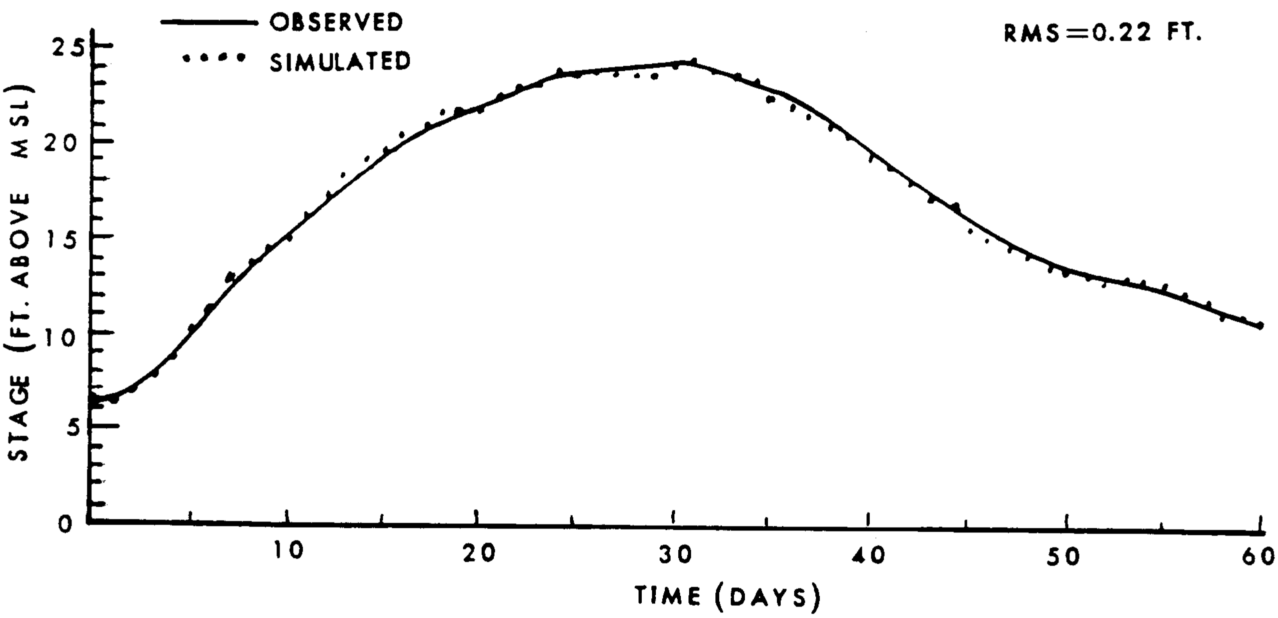


FIG. 3b - OBSERVED VS. SIMULATED STAGES AT DONALDSONVILLE FOR 1969 FLOOD

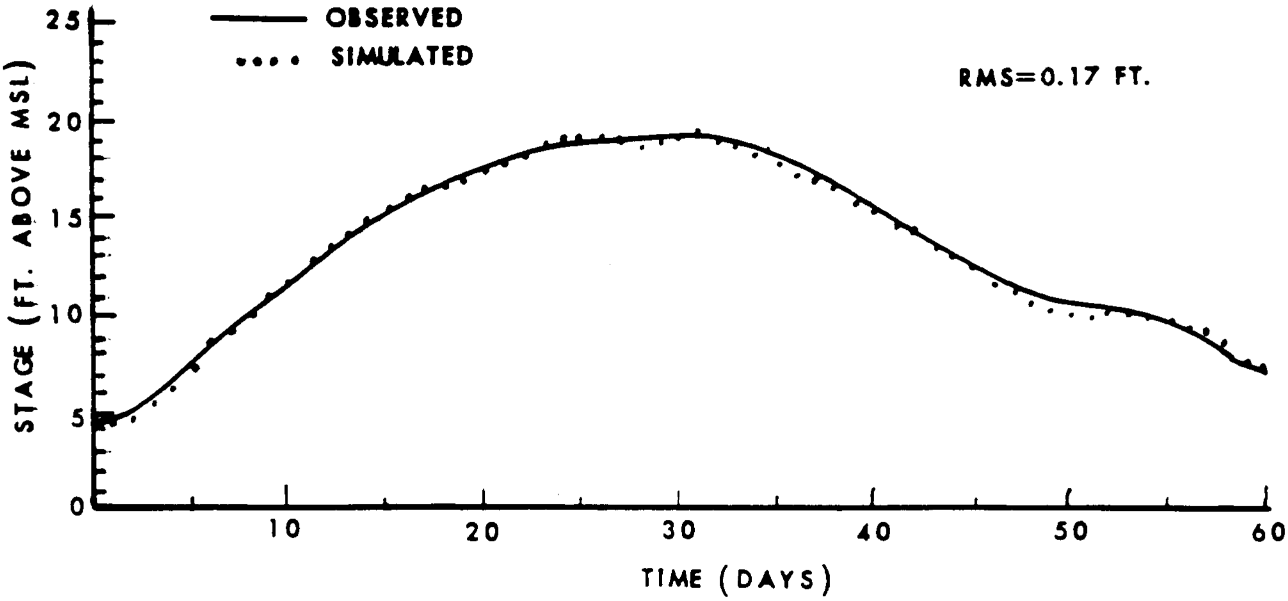


FIG.4a - OBSERVED VS. SIMULATED STAGES AT RESERVE FOR 1969 FLOOD

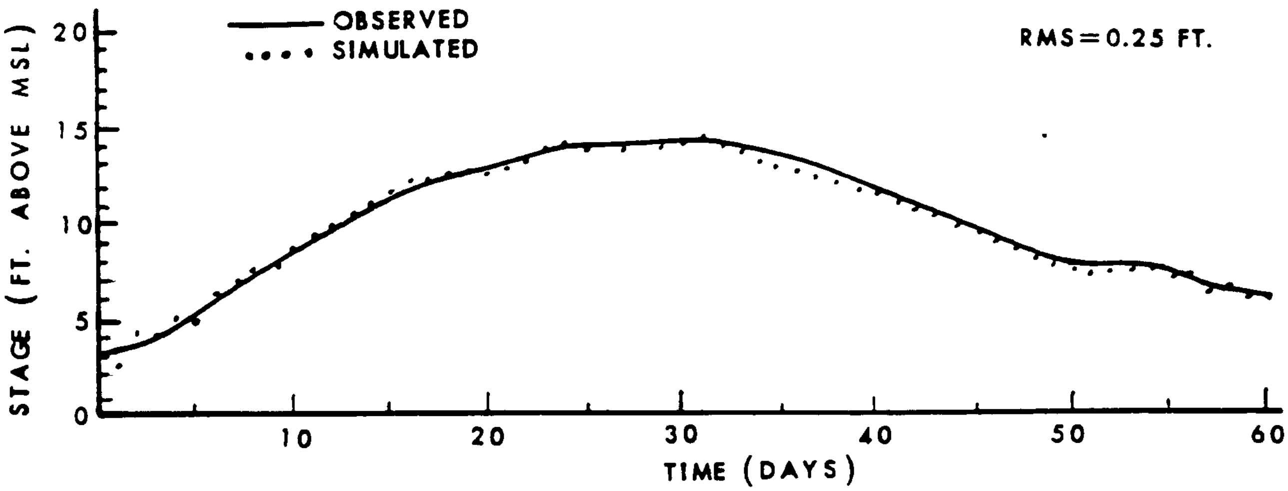


FIG.4b - OBSERVED VS. SIMULATED STAGES AT CARROLLTON (NEW ORLEANS) FOR 1969 FLOOD

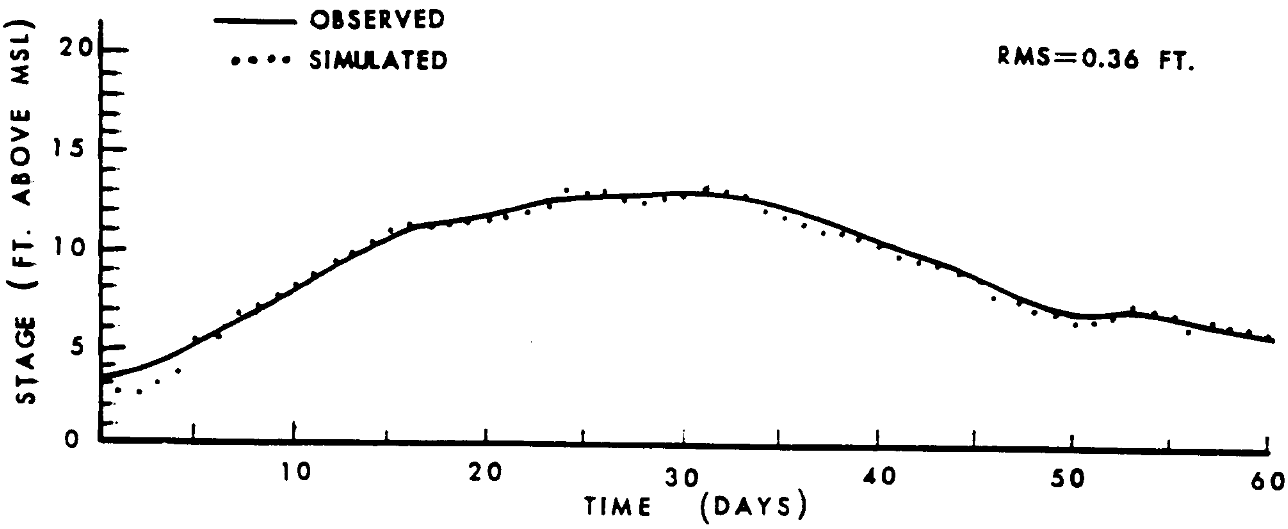


FIG. 5a - OBSERVED VS. SIMULATED STAGES AT CHALMETTE FOR 1969 FLOOD

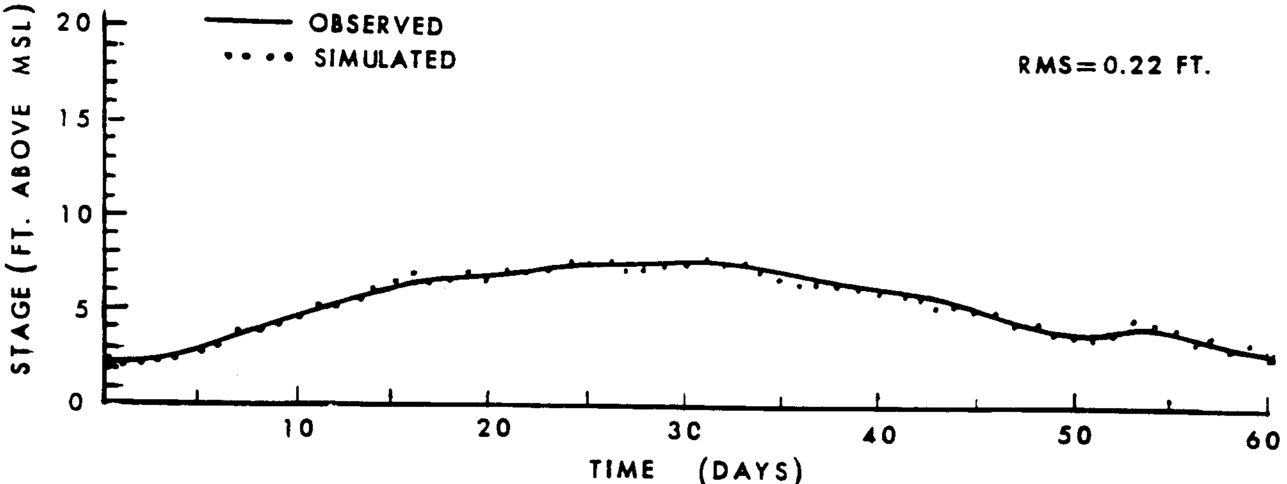


FIG. 5b - OBSERVED VS. SIMULATED STAGES AT PT. A LA HACHE FOR 1969 FLOOD

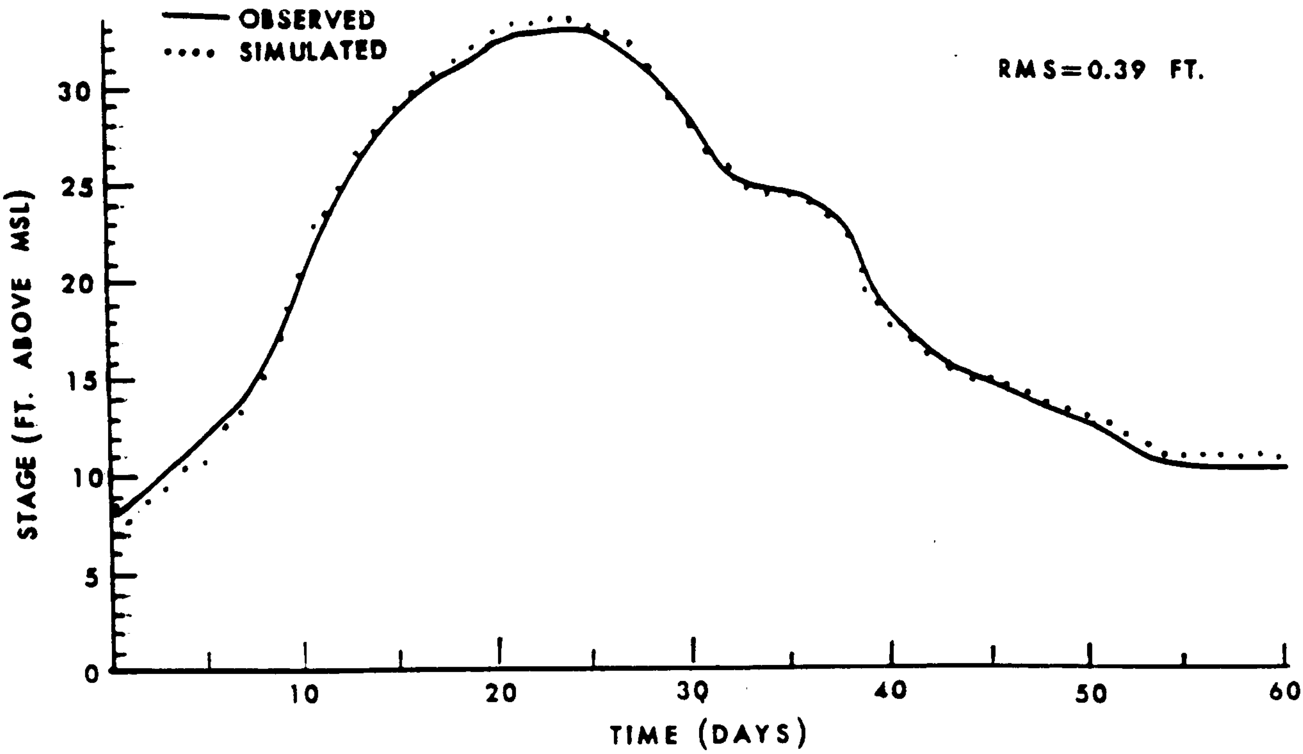


FIG.6a - OBSERVED VS. SIMULATED STAGES AT BATON ROUGE FOR 1966 FLOOD

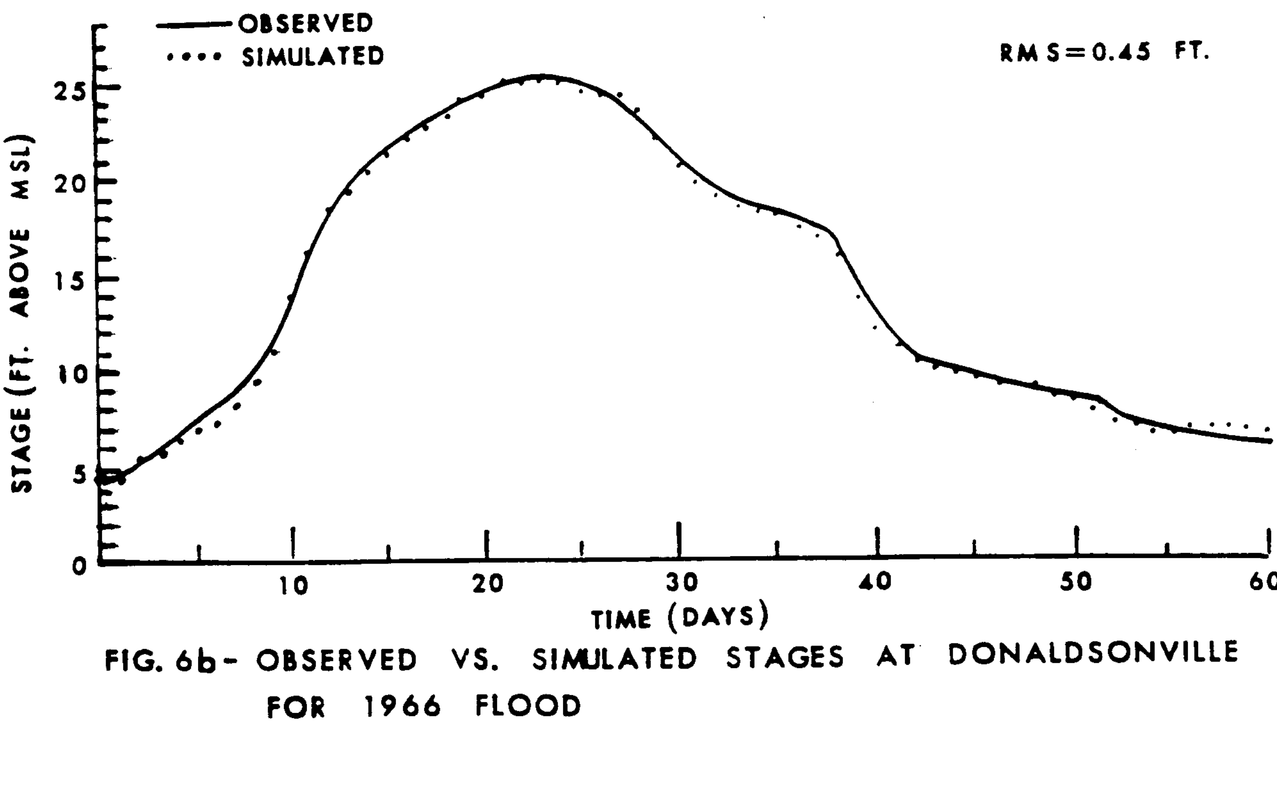


FIG. 6b - OBSERVED VS. SIMULATED STAGES AT DONALDSONVILLE FOR 1966 FLOOD

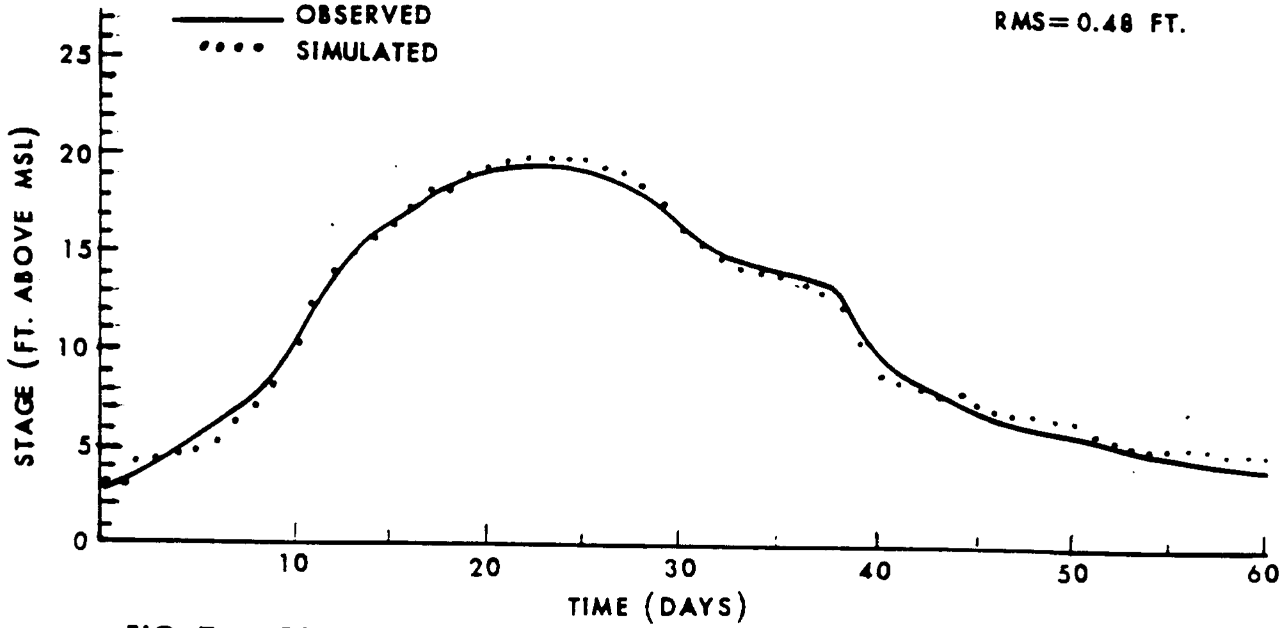


FIG. 7a - OBSERVED VS. SIMULATED STAGES AT RESERVE FOR 1966 FLOOD

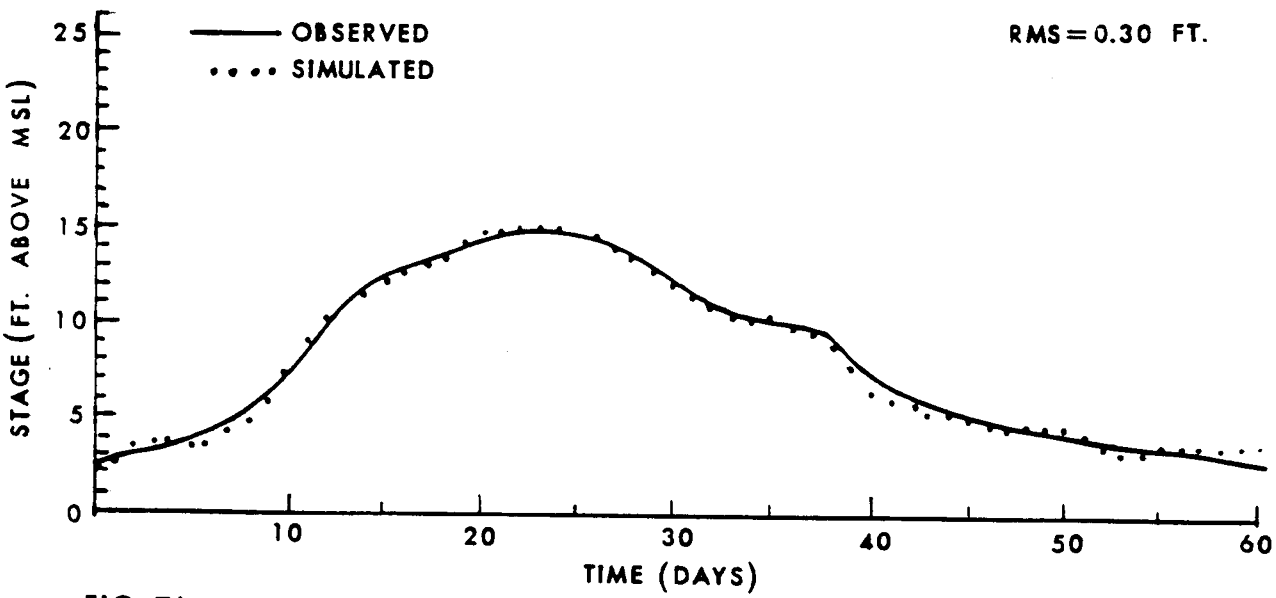


FIG. 7b - OBSERVED VS. SIMULATED STAGES AT CARROLLTON (NEW ORLEANS) FOR 1966 FLOOD

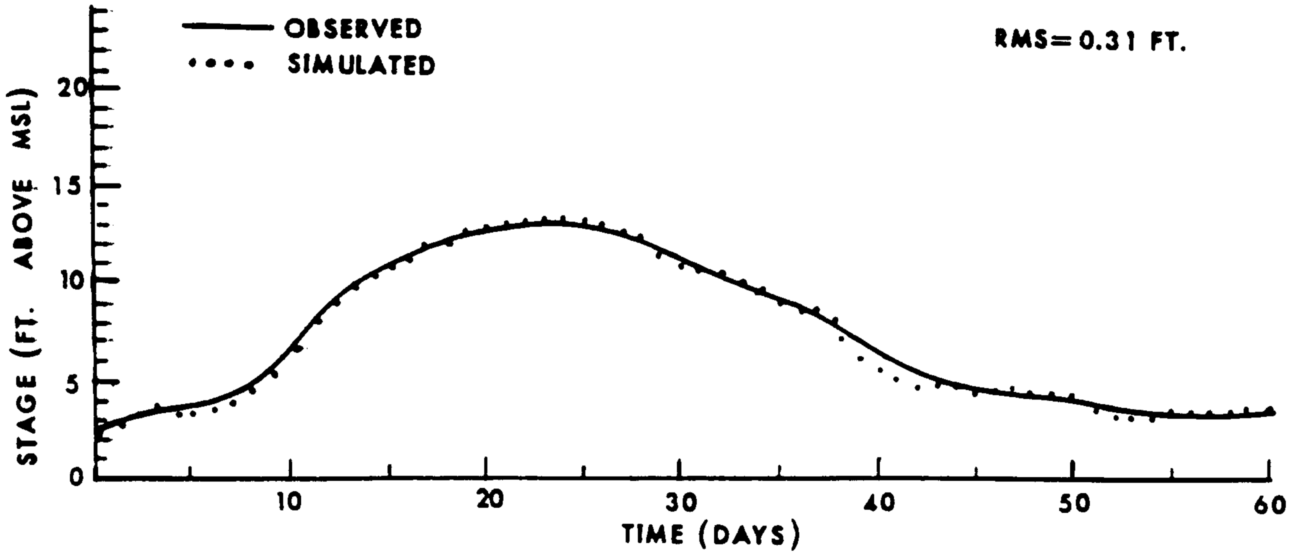


FIG. 8a- OBSERVED VS. SIMULATED STAGES AT CHALMETTE FOR 1966 FLOOD

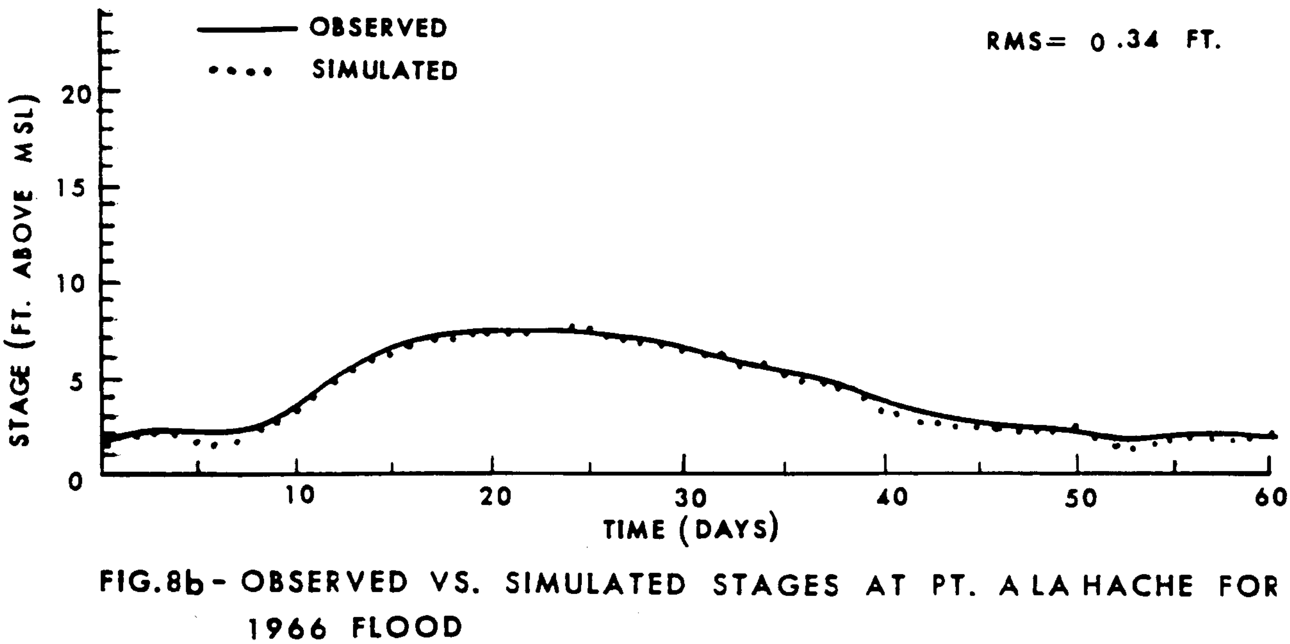


FIG.8b - OBSERVED VS. SIMULATED STAGES AT PT. A LA HACHE FOR 1966 FLOOD

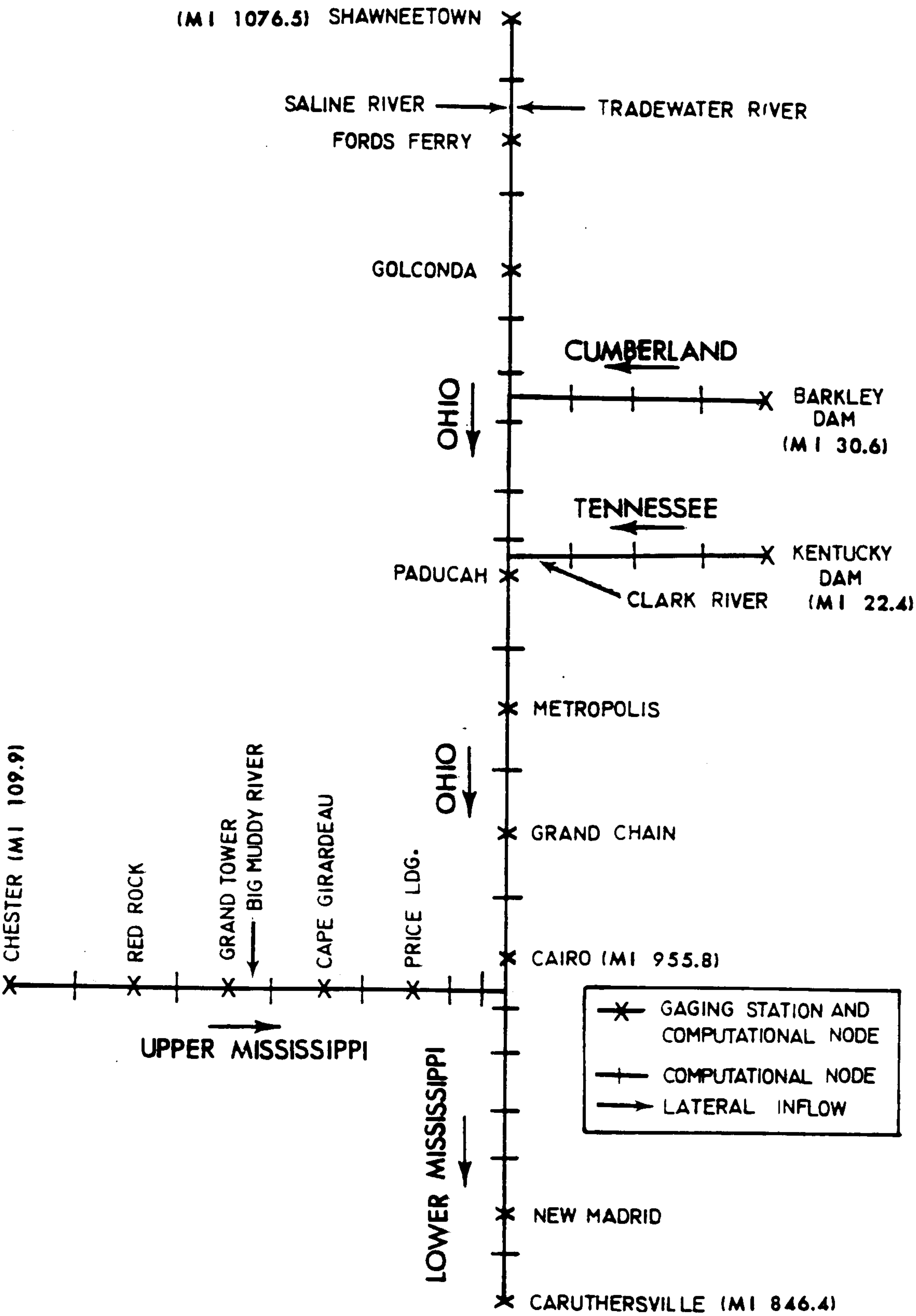


FIG. 9 - SCHEMATIC OF MISSISSIPPI-OHIO-CUMBERLAND-TENNESSEE (MOCT) RIVER SYSTEM

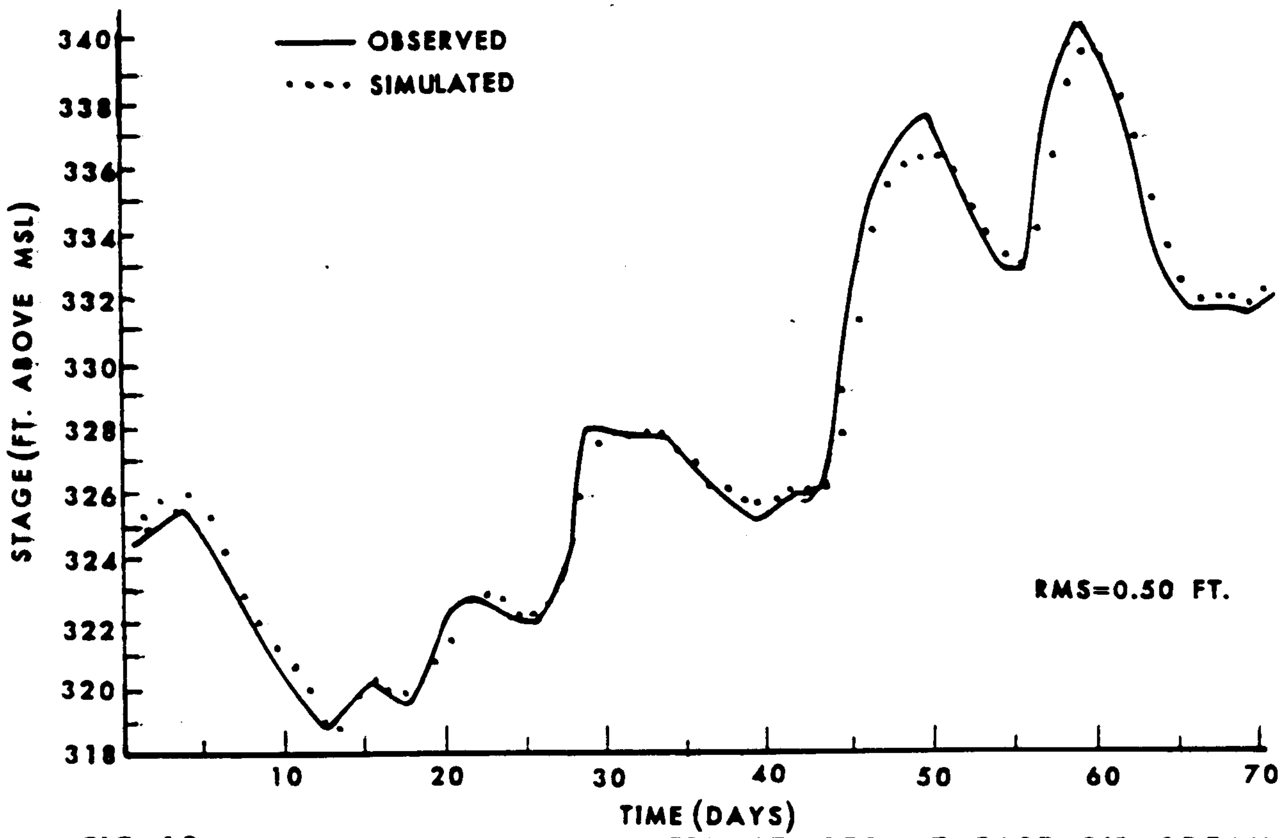


FIG. 10a - OBSERVED VS. SIMULATED STAGES AT CAPE GIRARDEAU FOR 1970 FLOOD

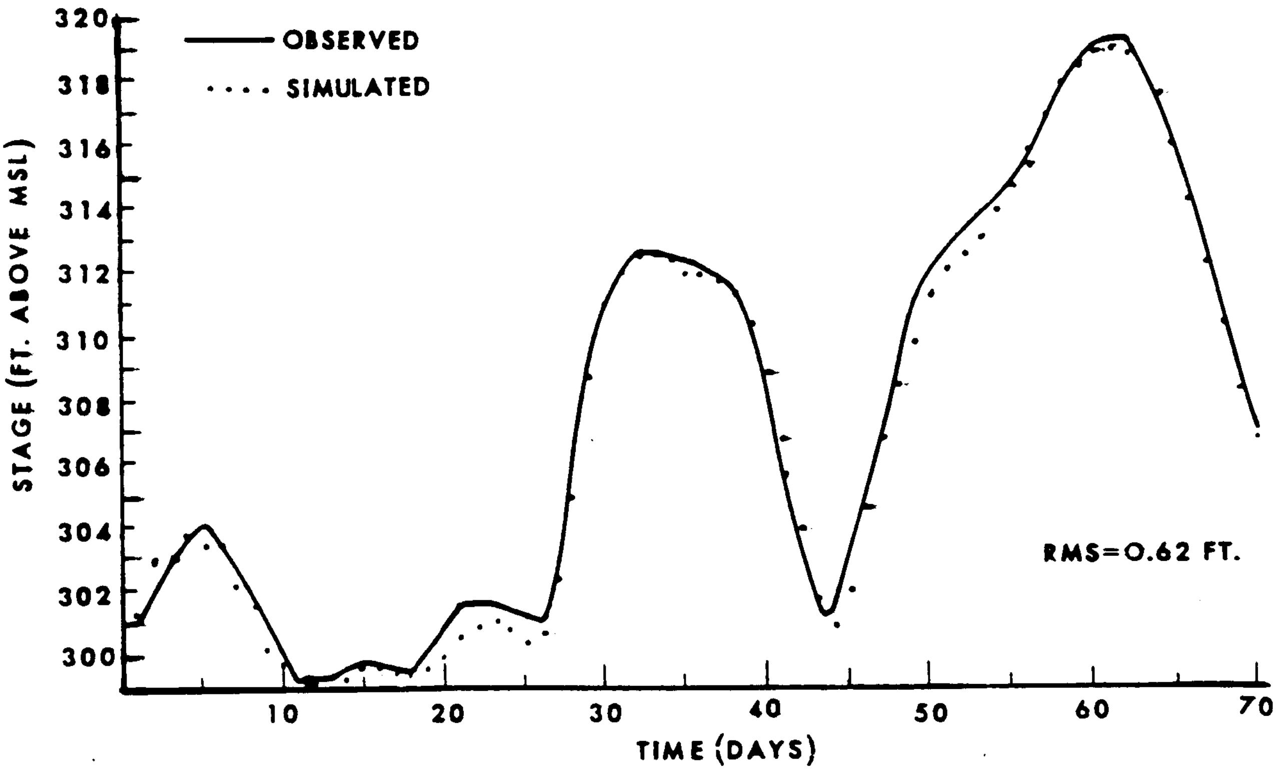


FIG. 10b - OBSERVED VS. SIMULATED STAGES AT CAIRO FOR 1970 FLOOD

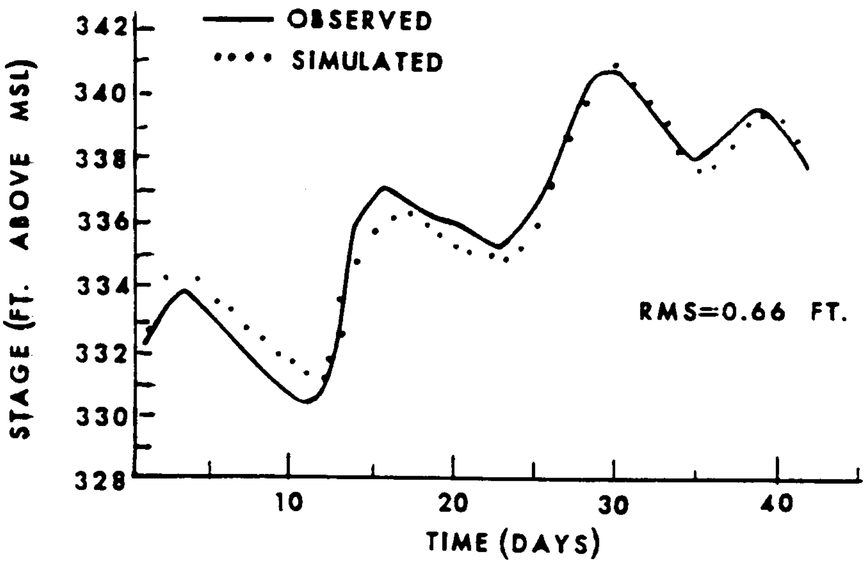


FIG. 11a - OBSERVED VS. SIMULATED STAGES AT CAPE GIRARDEAU FOR 1969 FLOOD

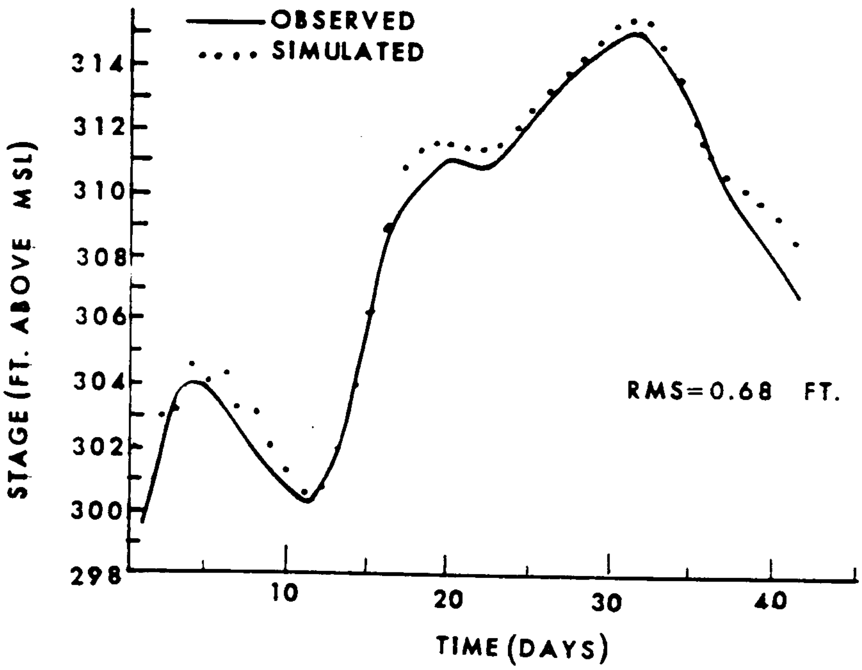


FIG. 11b - OBSERVED VS. SIMULATED STAGES AT CAIRO FOR 1969 FLOOD

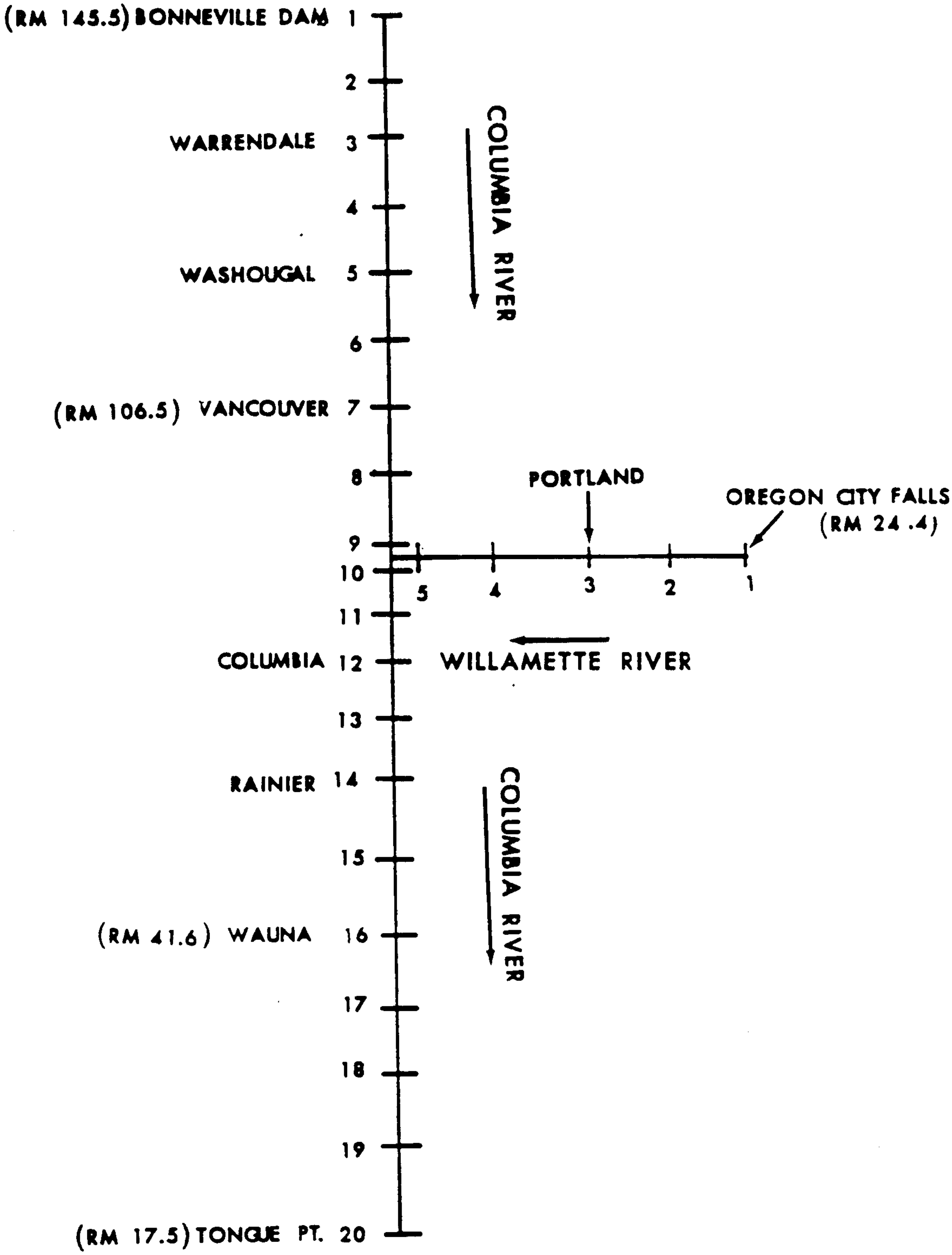


FIG. 12 - SCHEMATIC LOWER COLUMBIA-WILLAMETTE RIVER SYSTEM

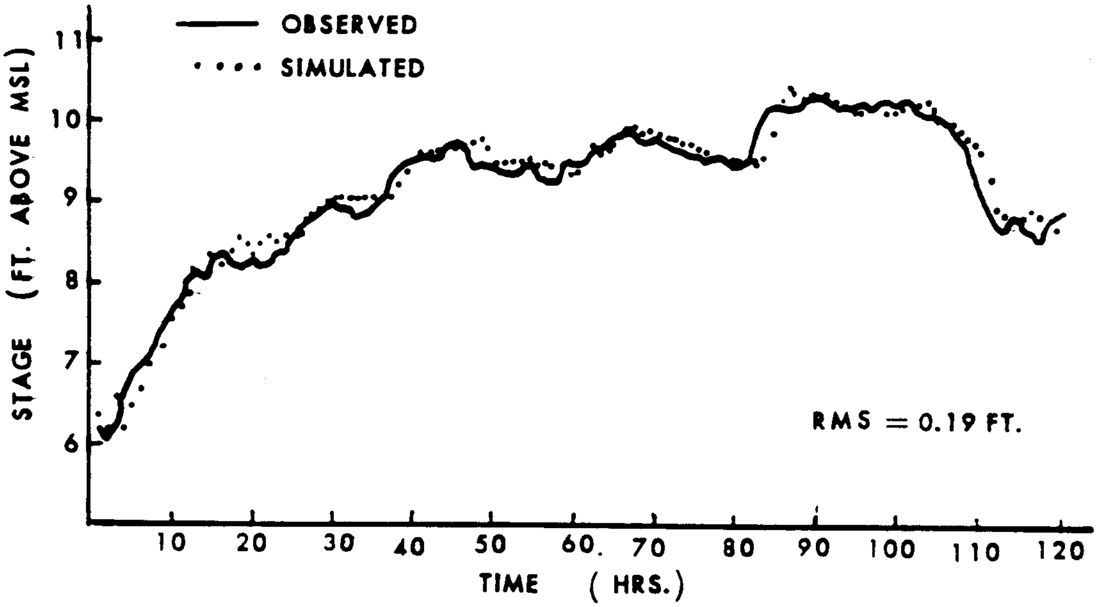


FIG. 13a - OBSERVED VS. SIMULATED STAGES AT WARRENDALE

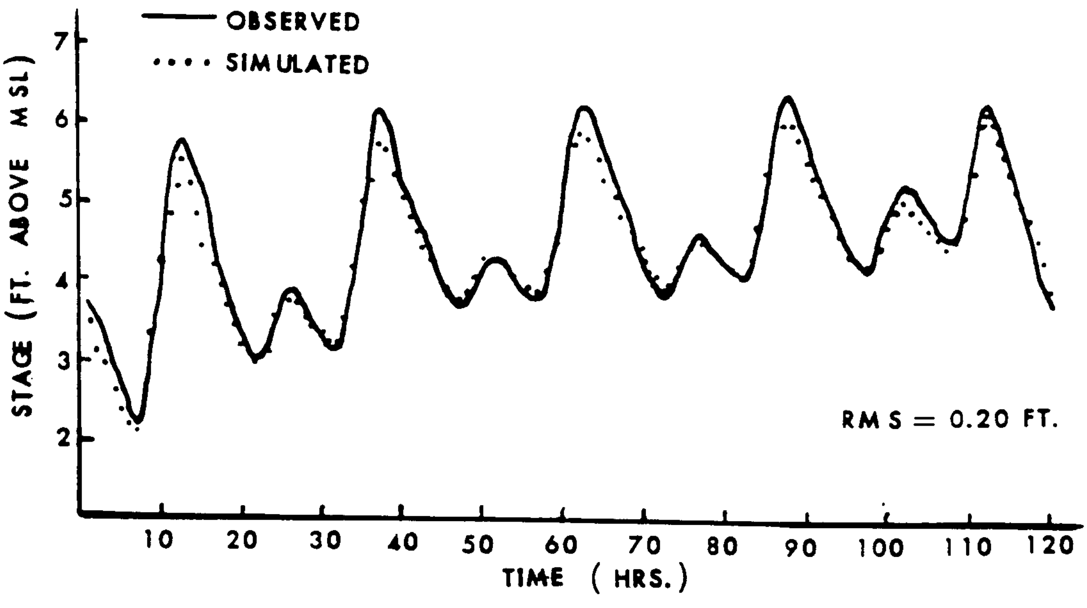


FIG. 13b - OBSERVED VS. SIMULATED STAGES AT VANCOUVER

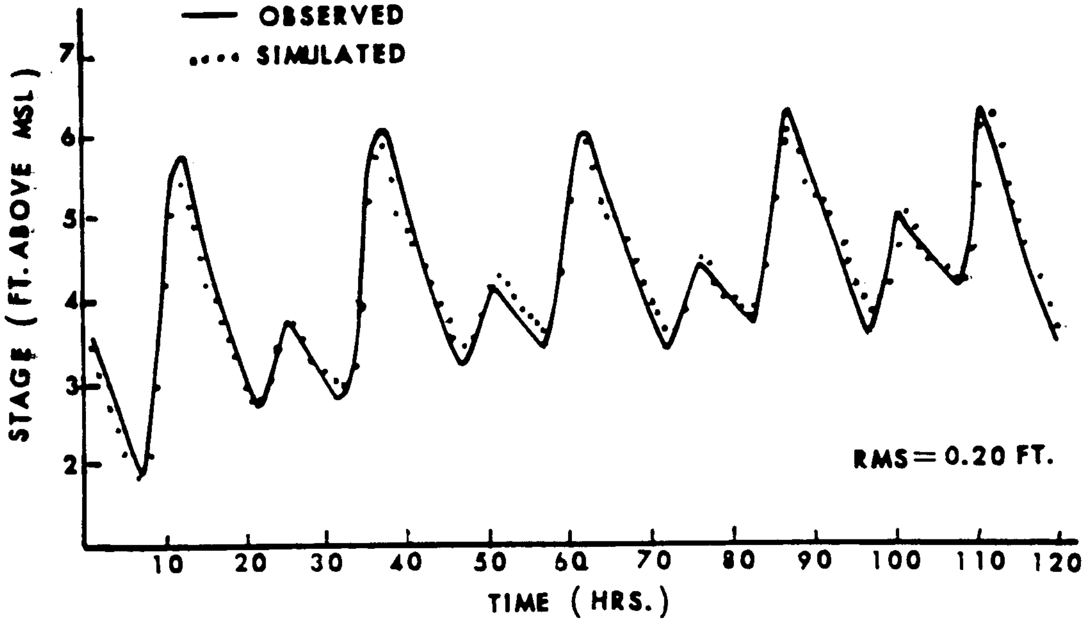


FIG. 14 a - OBSERVED VS. SIMULATED STAGES AT PORTLAND

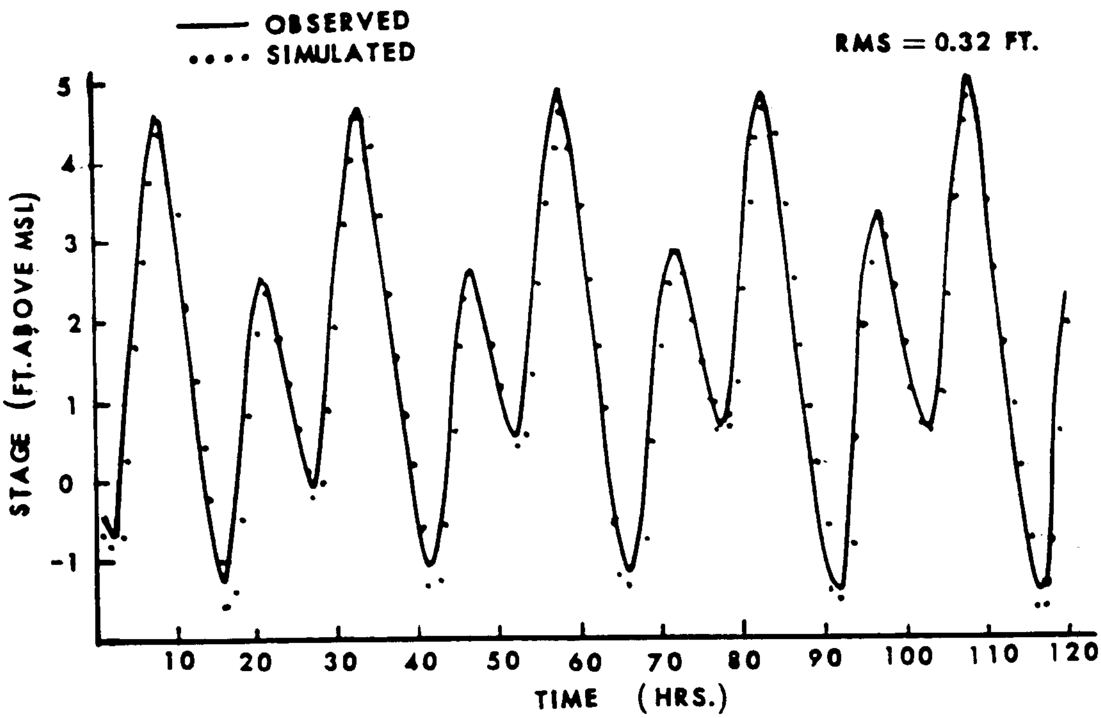


FIG. 14b - OBSERVED VS. SIMULATED STAGES AT WAUNA

APPENDIX A

INPUT DATA STRUCTURE FOR DYNAMIC WAVE MODEL

Version: 07/18/84

Card Input No.	Data Description - Input Format

(1) K1,K2,K3,K4,K5,K6,K7,K8,K9,K10,K11,K12,K13,K14 - 14I5
(See note at end of Data Input.)

(2) ALONE (Punch 'ALONE' starting in column 1.)

(3) EPSY,EPSQ,EPSQJ,THETA,F1,XFACT,DHF,CFNAME - 7F10.0,4X,A4

EPSY	Depth tolerance in Newton Iteration (0.001-1.0 ft). A good value is 0.05 ft.
EPSQ	Discharge tolerance in Newton Iteration (10.-10000. cfs). EPSQ=EPSY x avg. V x avg. width.
EPSQJ	Discharge tolerance in tributary iteration scheme (10.-10000.cfs). EPSQJ=EPSQ.
THETA	Acceleration factor in solving tributary junction problem (0.5-1.0). Varies with each problem with 0.8 a good first choice.
F1	θ weighting factor (normally F1=0.55, but can range from 0.5 to 1.0).
XFACT	Factor to convert units, describing the location of the computation points along the routing reach, to feet; e.g., if units are in miles, XFACT=5280.
DHF	Factor to convert time units associated with specified stage or discharge hydrograph data at the upstream or downstream boundaries into units of hours; e.g., if hydrograph data are read in as daily values, DHF=24.
CFNAME	Name identifying river system which when used in other than the ALONE command identifies carryover file in which this data is stored.

(4) JN,NU,NCT,ICD,NYQD,ITMAX,NCML - 8I10

JN	Number of rivers that are being routed simultaneously (no tributary, JN=1; 1 tributary, JN=2; 2 tributaries, JN=3).
NU	Number of values associated with specified stage or discharge hydrographs; if mathematical function is used to describe the specified hydrographs, NU=0.
NCT	Parameter indicating type of extrapolation used in Newton Iteration to determine estimates of unknowns; if extrapolation is parabolic, NCT=2; if extrapolation is linear, NCT=1; if no extrapolation, NCT=0.
ICD	If any boundary hydrograph changes by more than the value of this parameter (in ft) from the last time step, extrapolation is not used.
NYQD	Number of sets of stage-discharge values in empirical rating curve at downstream boundary.
ITMAX	Maximum number of iterations allowed in the Newton-Raphson Iteration Procedure for solving the system of nonlinear equations; if ITMAX=1, the nonlinear formuation degenerates into a linear formulation and no iterations are required in the Newton-Raphson Iteration Procedure.
NCML	Number of values in Manning's n versus stage or discharge table; this is the same for all Manning's n reaches.

(5) NCS,NCSS,NP,KTERM,KPL,KPL2,JNK,NPEND - 8I10

NCS	Number of values in table of top widths (BS) vs. elevations (HS) referenced to mean sea level (m.s.l.).
NCSS	Number of values in table of top widths of storage sections (BSS) vs. elevations (HSS) referenced to m.s.l.
NP	Parameter indicating if Automatic Calibration option is to be used; if NP=0, no Automatic Calibration will be made; if NP≥1, Automatic Calibration will be used; when using Automatic Calibration, NP also denotes the sequence number of the first computed stage (in the computed stage hydrograph) which will be used in the statistics needed in the automatic calibration for determining the Manning n. (See note at end of Data Input concerning type of data required for using Automatic Calibration option.)
KTERM	Parameter indicating if terms in equation of motion will be computed and printed as special information; if KTERM=1, they will be printed; if KTERM=0, they will not be printed.

KPL	Parameter indicating what information will be plotted; if KPL=0, nothing is plotted; if KPL=1, stage hydrographs are plotted; if KPL=2, discharge hydrographs are plotted.
KPL2	Parameter to denote if observed data are available for plotting at stations for which the computed results are to be plotted; if KPL=0, no data are available; if KPL2=1, observed data are available.
JNK	Parameter indicating if computed water surface elevations, velocities, and discharges will be printed; if JNK=0, they will <u>not</u> be printed; if JNK=1, they will be printed.
NPEND	Parameter indicating the last value in the computed stage hydrograph which will be used in the statistics needed in the Automatic Calibration option to determine the Manning n. If left blank, last value of observed stage hydrograph will be used.

(6) NQL(J) - 8I10

NQL(J)	Parameter for indicating if any lateral inflows are present; NQL(J)=0, if none are present; NQL(J)=K, where K is the number of Δx subreaches where lateral inflows are present; J index goes from 1 to JN.

(7) NWJ(J) - 8I10

NWJ(J)	Number of Δx reaches in J^{th} river where weir-flow (levee overtopping) may occur, J index goes from 1 to JN.

(8) NUMLAD(J) - 8I10

NUMLAD(J)	Number of locks and dams or internal boundaries in J^{th} river, J index goes from 1 to JN.

(9) NCSS1(J) - 8I10

NCSS1(J)	Number of stations that have off-channel storage.

If NCSS1(J)>0 (see card input no. 9), read in NCSSS(K,J)

See note following card 10

(10) NCSSS(K,J) - 8I10

NCSSS(K,J) Station number associated with the computational points or sections along the routing reach which have off-channel storage; computational points or sections are numbered from the upstream boundary toward the downstream boundary; K index goes from 1 to NCSS1(J) (number of stations that have off-channel storage).

Note: Repeat card 10 once for each J, J=1, JN.

(11) NB(1) - I10

NB(1) Number of stations or computational points along the main river.

If JN>1 (see card input no. 4), read in NB(J), NJUN(J), ATF(J).

See note following card 12

(12) NB(J),NJUN(J),ATF(J) - 2I10,F10.0

NB(J) Number of stations along the tributaries, where J is the number of tributary; tributaries are numbered from upstream to downstream along the main river commencing with the number 2.

NJUN(J) Number of station along the main river where tributary J enters (this station coincides with the upstream extremity of the small subreach which is equivalent in length to the tributary width).

ATF(J) Acute angle that the tributary makes with the main river at the confluence; ATF(J) is in degrees.

Note: Repeat card 12 for each J, J=2, JN.

(13) KU(J) - 8I10

KU(J) Parameter indicating the type of upstream boundary condition being specified for the main river or tributaries; if KU(J)=1, a stage hydrograph or if KU(J)=2, a discharge hydrograph is the upstream boundary condition; J index goes from 1 to JN.

(14) KD(J) - 8I10

KD(J) — Parameter indicating the type of downstream boundary condition being specified for the main river; if KD(J)=1, a stage hydrograph is the downstream boundary condition (in the case of tributaries, KD(J) where J goes from 2 to JN is always equal to 1) except for levee overtopping simulation in which overflow is ponded beyond levees (in this case KD(J)=2); if KD(1)=2, a discharge hydrograph is the downstream boundary condition; if KD(1)=3, a single value rating curve of discharge as a function of stage is the boundary condition; if KD(1)=4, a loop rating curve is the boundary condition; if KD(1)=5, normal flow computed from Manning's equation (with the channel bottom slope used as the energy slope) is the downstream boundary condition; if KD(1)=1 and NYQD>0, a single value rating curve in which Q is a function of the computed water surface minus the read-in value of STN.

See note following card 16

(15) NRCM1(J),NRT1(J),NNYQ(J) - 3I10

NRCM1(J) — Number of different Manning n relationships used in the routing reach (J^{th} river).

NRT1(J) — Total number of observed stage hydrographs along J^{th} river (routing reach) which will be compared with computed water surface elevations; maximum number that can be handled by the program is 9; also, denotes total number of stations for which computed values will be plotted.

NNYQ(J) — Parameter indicating if Manning's n is a function of water surface elevation or discharge; if NNYQ(J)=0, n is a function of water surface elevation; if NNYQ(J)=1, n is a function of discharge; if n is constant, let NNYQ(J)=0.

Note: If KD(1)=4 or KD(1)=5, add one to NRCM1(1) (for the main river only).

See note following card 16

(16) NCM(K,J) - 8I10

NCM(K,J) Station number of downstream most station in subreach that has the same Manning's n; K index goes from 1 to NRCM1(J).

Note: If KD(1)=4 or KD(1)=5, the last value of NCM for the main river should be one greater than the number of stations on the main river (NB(1)+1).

Note: Repeat card sequence 15 and 16 once for each J, J=1, JN.

If NRT1(J)>0 (see card input no. 15), read in NT(K,J).

See note following card 18

(17) NT(K,J) - 8I10

NT(K,J) Station number of each observed stage hydrograph; K index goes from 1 to NRT1(J).

If NRT1(J)>0 (see card input no. 15), and KPL=1 and KPL2=1 (see card input no. 5), read in GZ(K,J).

See note following card 18

(18) GZ(K,J) - 8I10

GZ(K,J) Gage correction to convert observed stage hydrographs to mean sea level datum; K index goes from 1 to NRT1(J).

Note: Read card sequence 17 and 18 once for each J, J=1, JN.

If NRT1(J)>0 (see card input no. 15), and KPL>0 (see card input no. 5), read in STTNAM(L,I,J).

See note following card 20

(19) STTNAM(L,I,J) - 5A4

STTNAM(L,I,J) 20-character name associated with observed stages (see card 20); L index goes from 1 to 5.

If NRT1(J)>0 (see card input no. 15) and KPL>0 and KPL2=1 (see card input no. 5), read in STT(K,I,J).

See note following card 20

(20) STT(K,I,J) - 8F10.0

STT(K,I,J) Observed stages; K index goes from 1 to NU.

Note: Repeat cards 19 and 20 once for each I, I=1, NRT1(J); and repeat this sequence for each J, J=1, JN.

If NU=0 (see card input no. 4), read in TP, RHO, GAMMA, YI.

(21) TP,RHO,GAMMA,YI - 4F10.0

TP	Time (hours) from initial steady flow to peak of specified upstream boundary hydrograph (used in mathematical function describing the hydrograph).
RHO	Ratio of peak value of specified hydrograph to initial value of hydrograph.
GAMMA	Ratio of time TG to TP, where TG is the time from inital steady flow to the center of gravity of the specified hydrograph (must be greater than 1.)
YI	Initial steady discharge or water surface elevation at upstream boundary.

If NU>0 (see card input no. 4), read in ST1(K,J).

See note following card 23

(22) ST1(K,J) - 8F10.0

ST1(K,J) Observed stages or discharges at upstream boundary of river J; K index goes from 1 to NU.

If NU>0 (see card input no. 4), and KU(J)=1 (see card input no. 13), read in GZ1(J).

See note following card 23

(23) GZ1(J) - 8F10.0

GZ1(J) Gage correction to convert observed stages at upstream boundary of river J to m.s.l. datum.

Note: Repeat card sequence 22, 23 once for each J, J=1, JN.

If KD(J)≤2 (see card input no. 14), read in STN(K,1)

(24) STN(K,1) - 8F10.0

STN(K,1) Observed stages or discharges at downstream boundary of main river; K index goes from 1 to NU with 8 values per data card.

If KD(J)=1 (see card input no. 14), or NYQD>0 (see card input no. 4), read in GZN.

(25) GZN - F10.0

GZN Gage correction to convert stages at downstream boundary of main river to m.s.l. datum.

If NYQD>0 (see card input no. 4), read in YQD(K).

(26) YQD(K) - 8F10.0

YQD(K) Stages used to define empirical rating curve at downstream boundary of main river; K index goes from 1 to NYQD.

If NYQD>0 (see card input no. 4), read in QYQD(K).

(27) QYQD(K) - 8F10.0

QYQD(K) Discharges used to define empirical rating curve at downstream boundary of main river; K index goes from 1 to NYQD.

See note following card 30

(28) BS(K,I,J) - 8F10.0

BS(K,I,J) Top widths of channel cross section at various elevations referenced to m.s.l. datum; K index goes from 1 to NCS; I index goes from 1 to NB(J); i.e., a set of BS top widths are read in for each station along river J.

See note following card 30

(29) HS(K,I,J) - 8F10.0

HS(K,I,J) Elevations (references to m.s.l. datum) corresponding to each top width (BS); K index goes from 1 to NCS; I index goes from 1 to NB(J). Elevations should proceed from bottom to top.

See note following card 30

(30) AS(1,I,J) - 8F10.0

AS(1,I,J) Channel cross sectional areas below HS(1,I,J) or lowest of the HS elevations; I index goes from 1 to NB(J).

Note: Repeat card sequence 28, 29, and 30 once for each J, J=1, JN.

If NCSS>0 (see card input no. 5), and NCSS1(J)>0 (see card input no. 9), read in BSS(K,I,J).

See note following card 33

(31) BSS(K,I,J) - 8F10.0

BSS(K,I,J) Top width of off-channel storage cross sections at various elevations referenced to m.s.l. datum; K index goes from 1 to NCSS.

Note: Only those stations with off-channel storage dimensions are read in; i.e., those stations which do not have any off-channel storage must not be included; a set of BSS top widths is read in for each station that has off-channel storage along the river corresponding to the J index. Stations having off-channel storage are defined by array NCSSS(L,J), where L=1, NCSS1(J).

If NCSS>0 (see card input no. 5), and NCSS1(J)>0 (see card input no. 9), read in HSS(K,I,J).

See note following card 33

(32) HSS(K,I,J) - 8F10.0

HSS(K,I,J) Elevations (references to m.s.l. datum) corresponding to each top width (BSS); K index goes from 1 to NCSS.

If NCSS>0 (see card input no. 5), and NCSS1(J)>0 (see card input no. 9), read n ASS(1,I,J).

See note following card 33

(33) ASS(1,I,J), 8F10.0

ASS(1,I,J) Off-channel storage cross-sectional area below HSS(1,I,J) or lowest of HSS elevations.

Note: Repeat card sequence 31, 32, and 33 for each I, I=1, NCSS(J), and repeat this sequence for each J, J=1, JN.

If KD(1)=5 (see card input no. 14), read in SO.

(34) SO - F10.6

SO — Average bottom slope of main river.

See note following card 37

(35) COFW,VWIND,WINAGL - 3F10.6

COFW — Coefficient of wind stress (1.1×10^{-6} to $3. \times 10^{-6}$).
VWIND — Wind velocity (ft/sec); + if directed upstream; - if directed downstream.
WINAGL — Acute angle (degrees) that wind makes with the channel axis.

See note following card 37

(36) X(I,J) - 8F10.2

X(I,J) — Location of station or cross section where computations are made (units can be anything since XFACT converts these units to ft); I index goes from 1 to NB(J).

See note following card 37

(37) FKC(I,J) - 8F10.0

FKC(I,J) — Expansion or contraction coefficients (expansion coefficients vary from -.30 to -1.00 and contraction coefficients vary from +.10 to +.40, depending on the sharpness of the transition section); if a station has no expansion or contraction, enter 0.0; I index goes from 1 to NB(J).

Note: Repeat card sequence 35, 36, and 37 for each J, J=1, JN.

If NQL(J)>0 (see card input no. 6), read in LQ(I,J).

See note following card 39

(38) LQ(I,J) - 8I10.0

LQ(I,J) — Number of station at upstream end of each Δx reach in which lateral inflow is introduced; stations are numbered from upstream to downstream extremities of the routing reach commencing with 1 and going to NB(J); I index goes from 1 to NQL(J).

If NQL(J)>0 (see card input no. 6), read in QL(K,I,J).

See note following card 39

(39) QL(K,I,J) - 8F10.0

QL(K,I,J) — Lateral inflow in cfs per Δx reach, either overland flow per Δx reach or tributary inflow per Δx reach; K index goes from 1 to NU; I index goes from 1 to NQL(J).

<u>Note</u>: Repeat card sequence 38 and 39 once for each lateral inflow point and each J, J=1, JN.

If NWJ(J)>0 (see card input no. 7), read in NWJX(K,J)

SEE NOTE FOLLOWING CARD 41

(40) NWJX(K,J) - 8I10.0

NWJX(K,J) — Number of station at upstream end of Δx reach in which weir-flow or levee overtopping and/or failure (crevasse) (inflow or outflow) may occur; stations are numbered from upstream to downstream extremities of routing reach commencing with 1 and going to NB(J); K index goes from 1 to NWJ(J).

If NWJ(J)>0 (see card input no.7) read in HWH(K),WC(K),TFL(K),BBL(K), HFL(K),HMINL(K)

See note following card 41

(41) HWH(K),WC(K),TFL(K),BBL(K),HFL(K),HMINL(K) - 6F10.2

HWH(K)	Elevation (ft m.s.l.) of top of levee, ridge line, etc. where weir-flow occurs; elevation is average throughout Δx reach where weir-flow occurs.
WC(K)	Weir-flow discharge coefficient for Δx reach where weir-flow (inflow or outflow) may occur; coefficient ranges from 2.6 to 3.2.
TFL(K)	Time (hr) from start of levee failure (crevasse) until the opening or breach is its maximum size.
BBL(K)	Final width (ft) of levee crevasse which is assumed to have a rectangular shape.
HFL(K)	Elevation (ft m.s.l.) of water surface when levee starts to fail.
HMINL(K)	Final elevation (ft m.s.l.) of bottom of levee crevasse.

Note: Repeat card 41 as K goes from 1 to NWJ(J). Cards 40 and 41 are read-in for main stem river (J=1) if NWJ(1)>0. Thereafter, only card 40 is read-in when NWJ(J)>0 as J goes from 2 to JN. If overtopping may occur and no crevasse is to be simulated, leave TFL,BBL,HFL, and HMINL blank.

(42) LAD(L,J) - 8I10

LAD(L,J)	Number of station at upstream end of Δx reach in which lock and dam or internal boundary is located; stations are numbered from upstream to downstream extremities of routing reach commencing with 1 and going to NB(J); L index goes from 1 to NUMLAD(J). Use (-) integer value to activate internal boundary capability.

(43) POLTAR(L,J) - 8F10.2

POLTAR(L,J)	Elevation (m.s.l.) of water surface in headwater pool at upstream face of lock and dam; this elevation is considered the target pool elevation; the lock-master controls the flow through the dam via gates to maintain the pool elevation at this target elevation; L index goes from 1 to NUMLAD(J). Also, target discharge (cfs) at which gate rating changes to tailwater rating (for Ohio River).

(44) CHCTW(L,J) - 8F10.2

CHCTW(L,J) Elevation (m.s.l.) of water surface in tailwater pool at downstream face of lock and dam; this elevation is considered the elevation at which the lock master can no longer control the flow through the dam and the flow becomes channel control; usually this elevation will be equal or slightly less than the target pool elevation; L index goes from 1 to NUMLAD(J). Also, specifies type of internal boundary condition (See internal boundary note on page A-18 for additional types of internal boundary conditions that can be simulated.)

(45) RHI(K,L,J) - 8F10.2

RHI(K,L,J) Stage (ft m.s.l.) for internal boundary rating curve; omit unless LAD(I,J)<0 and CHCTW(L,J)≤0.05; L goes from 1 to NUMLAD(J); K goes from 1 to 8.

(46) RQI(K,L,J) - 8F10.2

RQI(K,L,J) Discharge (cfs) corresponding to RHI(K,L,J) or bridge embankment flow coefficient if CHCTW(L,J)=0.05. Omit if RHI (K,L,J) is omitted.

(47) GFR(K,L,J) - 8F10.2

GFR(K,L,J) Gate setting (horizontal opening of gate (ft), if CHCTW(L,J)=0.01; differential head if CHCTW(L,J)=0.03); omit unless LAD(I,J)<0 and 0.01≤CHCTW(L,J)≤0.03; L goes from 1 to NUMLAD(J); K goes from 1 to 8.

(48) GQ(K,L,J) - 8F10.2

GQ(K,L,J) Gate discharge (cfs) corresponding to GFR(K,L,J). Omit if GFR(K,L,J) is omitted.

(49) GFT(K,L,J) - 8F10.2

GFT(K,L,J) Gate setting (horizontal ft. of gate opening) time series; L goes from 1 to NUMLAD(J); K goes from 1 to NU. Omit unless GFR(K,L,J) is read-in or unless CHCTW(L,J)=0.05. In the latter, K goes from 1 to 3.

(50) POOLT(K,L,J) - 8F10.2

POOLT(K,L,J) Target pool elevation (same as POLTAR(L,J)) for each time step; if 0.0 or blank is read in, then POLTAR(L,J) is used for POOLT(K,L,J); K index goes from 1 to NU; L index goes from 1 to NUMLAD(J).

(51) ITWT(K,L,J) - 8I10.0

ITWT(K,L,J) Parameter indicating if gates control the flow; if 0 or blank, flow is controlled by gates, if 1, flow is not controlled by gates, e.g., the entire dam is removed as in the case of the low lift dams on the Lower Ohio River and the flow becomes channel controlled; K index goes from 1 to NU; L index goes from 1 to NUMLAD(J).

Note: Cards 42-51 are read in for each river as J goes from 1 to JN if NUMLAD(J)>0.

(52) TE,TM,KITPR - 2F10.2,I10

TE Time (hours) at which routing computations will terminate.

TM Maximum size Δt computational time step (hours).

KITPR Number of Δt time step intervals at which computed values are stored for plotting or printed out. Length of time step times KITPR equals interval at which plots are generated, e.g., if 6-hr time steps are taken and a plot is desired each day, set KITPR=4.

(53) YD1(I,J) - 8F10.0

YDI(I,J) Initial water surface elevations referenced to m.s.l. datum at each station; I index goes from 1 to NB(J); if blank or zeros are used for all the YDI(I,J) values, the program will generate the YDI's via linear interpolation between gaging stations (this is allowed when using the Automatic Calibration option or when gaging stations exist at the upstream extremities of all rivers and the downstream extremity of the main stem); if the upstream extremity does not have an observed hydrograph, this YDI value must be supplied along with all the zeros for the other YDI's; if zeros or blanks are used for all the YDI's except at the downstream extremity of the main stem river when the actual YDI is read in, the program will generate the YDI's via a solution of the steady flow backwater equation.

(54) QDI(I,J) - 8F10.0

QDI(I,J) Initial discharge (cfs) at each station; I index goes from 1 to NB(J); if blank or zeros are used for all the QDI (I,J) values except at the upstream extremity of each river, the program will generate the QDI's by summation of the flows from the upstream to downstream boundaries, including tributary inflow to the main stem and lateral inflow occurring along either the main stem or tributaries.

Note: Repeat card 53 for each J, J=1, JN, and then repeat card 54 for each J, J=1, JN.

See note following card 56

(55) YQCM(L,K,J) - 8F10.0

YQCM(L,K,J) Water surface elevations referenced to m.s.l. datum or discharges associated with n; L index goes from 1 to NCML.

Note: Repeat card 55 for each K, K=1, NRCM1(J); repeat sets of card 55 for each J, J=1, JN.

See note following card 56

(56) CM(L,K,J) - 8F10.0

CM(L,K,J) Manning's n; L index goes from 1 to NCML corresponding to each YQCM(L,K,J) value.

Note: Repeat card 56 for each K, K=1, NRCM1(J); repeat sets of card 56 for each J, J=1, JN.

Note: If KD(1)=4 or KD(1)=5, the values for YQCM(L,NRCM1(1),1) and CM(L,NRCM1(1),1) (the last values for the main river) are used in the loop rating curve or Manning's equation to compute the downstream boundary condition.

(57) MESAGE - 20 A 4 Format

MESAGE — Up to an 80-character message which will be written at the end of the input data deck.

(58) END - A 4 Format

END — Indicates end of input data deck.

(59) EXIT (Punch 'EXIT' starting in column 1.)

Automatic Calibration Note:

The following data must be included when using the Automatic Calibration option: Upstream boundary condition consisting of a discharge hydrograph for each river, i.e., KU(J)=2; observed stage hydrographs STT(K,I,J) at upstream and downstream ends of each reach wherein the Manning n is to be determined; and a downstream boundary condition consisting of a stage hydrograph, i.e., KD(J)=1; (the stage hydrograph STN(K,I,J) is required for the main stem river, only, since the model can provide the downstream stage hydrograph for each tributary via linear interpolation between observed stage hydrographs along the main stem which are adjacent to the tributary junction).

Plotting Note:

To obtain plots when no observed data are available, set KPL2=0, KPL=1 or 2, NRT1(J)>0, NT(K,J)>0, and enter STTNAM(L,I,J) for each station to be plotted.

Interior Drainage Note:

Overtopping of levees with the resulting flow ponded within the confines of a blocked tributary can be simulated by using cards 40-41 and a value of (2) for KD(2) on card 14. Also required is a tributary having a continuous low flow discharge and a large overbank and off-channel storage area representing the storage in the ponded area outside the levees. Interior drainage back into the river through a gravity-controlled check valve and pipe can also be simulated in the same manner, except HWH (card 41) is the pipe invert elevation and WC (card 41) is computed as 6.4 x pipe cross-sectional area ft^2). Of course, the Δx reach in which the pipe extends through the levee should be a relatively short reach (say 50 to 100 ft). If the receiving tributary of the overtopping levee flow is hydraulically connected to the main river, KD(2) on card 14 should have a value of 1. If WC (card 41) is entered as a negative value, the gravity check value is assumed to not exist therefore allowing flow to and from the river.

INTERNAL BOUNDARY NOTE

Various types of internal boundaries may be simulated by entering a specific value for CHCTW(L,J) on Card No. (44). The types are:

CHCTW(L,J)	REMARKS
0.00	Rating curve, $Q_i=f(h_i)$ where h_i is the water surface elevation at the upstream face of the internal boundary reach.
0.01 (Moveable Gate) (Ohio River)	Rating curve, $Q_i=f(h_{i+1})$ where h_{i+1} is the water surface elevation at the downstream face of the internal boundary reach. This holds as long as Q_{i+1}>POLTAR(L,J); otherwise, a gate rating curve GQ(K,L,J)=f(GFR(K,L,J)) where the value for determining the discharge from the rating curve is obtained from the time series GFT(K,L,J) which is the gate setting vs. time. GFR(K,L,J) is the gate opening in ft. of gate. (This option is for the Ohio River.)
0.03 (Siphon)	Same as for 0.01 except the value for determing the discharge from the rating is the differential head (h_i-h_{i+1}). A siphon rating or other structure with a constant flow area can be simulated by letting GFT(K,L,J)=1.0 and entering zero values for the GQ and GFR arrays with POLTAR(L,J) entered as zero.
0.05 (Bridge)	Bridge internal boundary with contracted flow $[Q_i=\sqrt{2g}c_cA_{i+1}(h_i-h_{i+1})^{1/2}]$ or orifice flow $[Q_i=\sqrt{2g}c_oA_{i+1}(h_i-h_{i+1})^{1/2}]$ and broad-crested weir flow over the road embankment including the bridge deck $[Q_i=S_bc_e(h_i-h_e)^{3/2}]$ where c_c=contracted opening flow coefficient, A_{i+1}=area of flow at downstream end of bridge opening, c_o=orifice flow where $h_i>h_b$ in which h_b=elev. of bridge deck, c_e=coefficient of flow for broad-crested weir multiplied by the length of the weir-crest (c_e is specified via RQI(K,L,J) as a function of h_i or RHI(K,L,J), S_b=submergence correction factor = $1.0-27.8(r-0.67)^3$ where $r=(h_{i+1}-h_e)/(h_i-h_e)$ in which h_e is the weir crest elev. which is specified by RHI(1,L,K). c_c,c_o,h_b are specified by GFT(1,L,J), GFT(2,L,J), GFT(3,L,J), respectively (only 3 values of GFT are read in).
>0.1 (Tide Gate)	Tide gate flow $[Q_i=c(h_i-h_{i+1})^{1/2}]$ where c is the discharge coefficient given by CHCTW(L,J) which is computed as $(2g)^{1/2}$ x discharge loss coef. x max. area of tide gate(s). If $h_{i+1} \geq h_i$, then Q_i=0.

K DIMENSIONAL VALUES

The most recent version of the DWOPER is programmed with a feature called variable dimensioning. This means that the size of subscripted variables can be changed from one execution of the DWOPER to another. There is a maximum total size for the sum of all arrays, but within that bound the allocation of space among variables is flexible.

For example, when the Ohio-Mississippi Junction (OMJ) is simulated, space must be available for four rivers, with a maximum of 31 computational points on a river. The same space can also be allocated so that for the Lower Mississippi, where only one river is simulated, 30 computational points can be used. Actually, the space for the Lower Mississippi is still less than for the OMJ so, for instance, more time steps could be used.

The dimensions of all arrays are set at the start of each execution of the DWOPER. A single card with 14 values (K1 through K14) sets the maximum dimensions needed. The dimensions which must be set at the start of each execution are:

K1 The number of rivers

K2 The maximum number of computational points on any river

K3 The number of time steps

K4 The maximum number of gaging stations on any river

*K5 The maximum number of HO values on any river

K6 The number of values in the downstream rating curve

K7 The maximum number of Manning n reaches on any river

K8 The maximum number of Manning n values in any Manning n reach

K9 The maximum number of water surface elevation vs. top width values at any cross-section

K10 The maximum number of lateral inflows on any river

*K11 The maximum number of sets of initial conditions to be stored in the carryover file (set to 5)

*K12 The maximum number of computed hydrographs to be saved after simulation

*K13 The total number of stage and discharge hydrographs in any hydrograph file which will be accessed

K14 The maximum number of water surface elevation vs. top width values for any off-channel storage

*Set these values to one except when using NWS operational program.

APPPENDIX B

ADDITIONAL INPUT DATA FOR "NETWORK, Critical Flow and Time-Dependent Gate" OPTIONS in DWOPER

Version: 1/3/86

Card Input No.	Data Description - Input Format

(1) K1,K2,K3,K4,K5,K6,K7,K8,K9,K10,K11,K12,K13,K14,K17,K18 - 16I5

K17 Total number of upstream boundaries in the system.

K18 Total number of junctions in the system.

If NETWORK option is not used, leave K17,K18 blank.

(1A) K21,K22 - 2I5 format

K21 Maximum number of gates in the system.

K22 Maximum number of values in the gate coefficient table.

(3) EPSY,EPSQ,EPSQJ,THETA,F1,XFACT,DHF,CFNAME - 7F10.2,4X,A4

CFNAME If = IMET, the metric option is used. With this option, HGC on card no. 46C and HG on card no. 46D are input in metric; and all output data is printed in metric units.

(4) JN,NU,NCT,ICD,NYQD,ITMAX,NCML,NET - 8I10

NET Parameter indicating if Channel Network option is to be used. If NET=0, this option is not used; if NET≥1, this option is to be used and JN should be set equal to 1. If NET=1, junction losses due to mixing and friction are neglected. These are included if NET=2.

(8A) NG(J) - 8I10

NG(J) Number of gates and/or critical flow sections in J^{th} river, J index goes from 1 to JN.

If NET≥1 (see card input no. 4), read in JCOD(K).

(11A) JCOD(K) - 8I10

JCOD(K) Code denoting the type node at each cross-section. JCOD(K) can have values of 1,2,3,...,11. K index goes from 1 to NB(J) (See Card 11).

If NET≥1 (see card input no. 4), read in JTN, NUPB

(11B) JTN,NUPB - 2I10

JTN Total number of junctions in the system.
NUPB Total number of upstream boundaries in the system.

If NET≥1 (see card input no. 4), read in JU(K), JB(K), JL(K).

See note following card 11C

(11C) JU(K), JB(K), JL(K) - 3I10

JU(K) Sequence number of cross-section at entry to junction.
JB(K) Sequence number of cross-section at branch of junction.
JL(K) Sequence number of cross-section at exit of junction.

Note: Omit card 11C if JTN=0

Note: Repeat card 11C for each junction.

If NET ≥1 (see card input no. 4) read in KMU(K).

(11D) KMU(K) - 8I10

KMU(K) Parameter indicating the type of upstream boundary condition: if KMU(K)=1, a stage hydrograph is the boundary condition; if KMU(K)=2, a discharge hydrograph is the boundary condition; or if KMU(K)=4, a depth-discharge relation is the boundary condition. K index goes from 1 to NUPB.

Note: When 11D is used, omit card 13.

(11E) JCPR(K) - 8I10

JCPR(K) Parameter indicating sequential order that cross sections will be listed in output, K index goes from 1 to NB(J).

If NG(J)>0 (see card input no. 8A) read in LGX(K,J), IGT(K,J), NTG(I,K), HGC(K,J).

See note following card 51D

(14) KD(J) - 8I10

KD(J) .....If KD(1)=6, downstream boundary is critical flow equation unless read-in value of (STN+GZN) is greater, in which case the latter is used.

(51A) LGX(K,J),IGT(K,J),NTG(K,J),HGC(K,J) - 3I10,F10.2

LGX(K,J) Sequence number of cross section immediately upstream of gate internal boundary.

IGT(K,J) Parameter indicating the type of gate: if IGT(K,J)=1, the gate has a continuous bottom slope; if IGT(K,J)=2, the gate has a discontinuous bottom slope; if IGT(K,J)=0, the location is a critical flow section.

NTG(K,J) Number of values in the table of gate coefficient (GC) vs. tailwater head (DTW).

HGC(K,J) Gate correction parameter used to correct the height of the gate opening.

If NG(J)>0 (see input no. 8A) read in GC(I,K,J).

See note following card 51D

(51B) GC(I,K,J) - 8F10.2

GC(I,K,J) Gate discharge coefficient; coefficient is determined by multiplying the width of the gate by the loss factor; I index goes from 1 to NTG(K,J).

If NG(J)>0 (see card input no. 8A) read in DTW(I,K,J).

See note following card 51D

(51C) DTW(I,K,J) - 8F10.2

DTW(I,K,J) Tailwater head (tailwater elevation - gate bottom elevation) corresponding to each gate discharge coefficient; I index goes from 1 to NTG(K,J)

If NG(J)>0 (see card input no. 8A) read in HG(I,K,J).

See note following card 51D

(51D) HG(I,K,J) - 8F10.2

HG(I,K,J) Height of gate opening; I index goes from 1 to NU.

Note: Omit card sequence 51B, 51C, and 51D if IGT(K,J) = 0.

Note: Repeat card sequence 51A, 51B, 51C, and 51D once for each K, K=1,NG(J), and repeat this procedure for each river as J goes from 1 to JN.

(54A) NR - I10

NR Number of separate channels or branches or storm sewers in the system.

(54B) NIRB(K) - 8I10

NIRB(K) Number of nodes in each channel or branch; K index goes from 1 to NR; the number of nodes along the main channel or sewer is entered first; then the number of nodes in the most upstream branch is entered with the next most upstream branch following until the most downstream branch is entered.

(54C) IRB(I,K) - 8I10

IRB(I,K) Sequential number of node in each branch; node numbers commence at upstream end of each branch and proceed in the downstream direction until the last node of the branch is reached; I index goes from 1 to NIRB(K).

Note: Repeat card 54C as the K index goes from 1 to NR; branches are ordered from upstream to downstream and the first entry is the main (principal) channel or storm sewer.

Note: Cards 54A,B,C are omitted if NET = 0, or QDI(2,1) not equal to zero and NET ≥ 1.

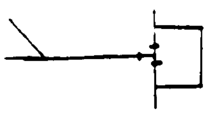
NODE CODE (JCOD(K), K=1,N)

1 Internal Node

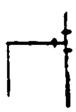
2 Upstream Boundary

3 Downstream Boundary

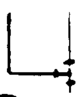
4 Tributary Junction



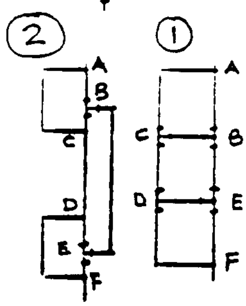
5 Tributary Junction on a Bifurcation



6 Upstream Bifurcation Junction

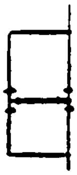


7 Downstream Bifurcation Junction



8 Double Bifurcation Junction on a Bifurcation

Configuration 1 must be converted to 2.



9 Single Bifurcation Junction on a Bifurcation

10 Gate Internal Boundary

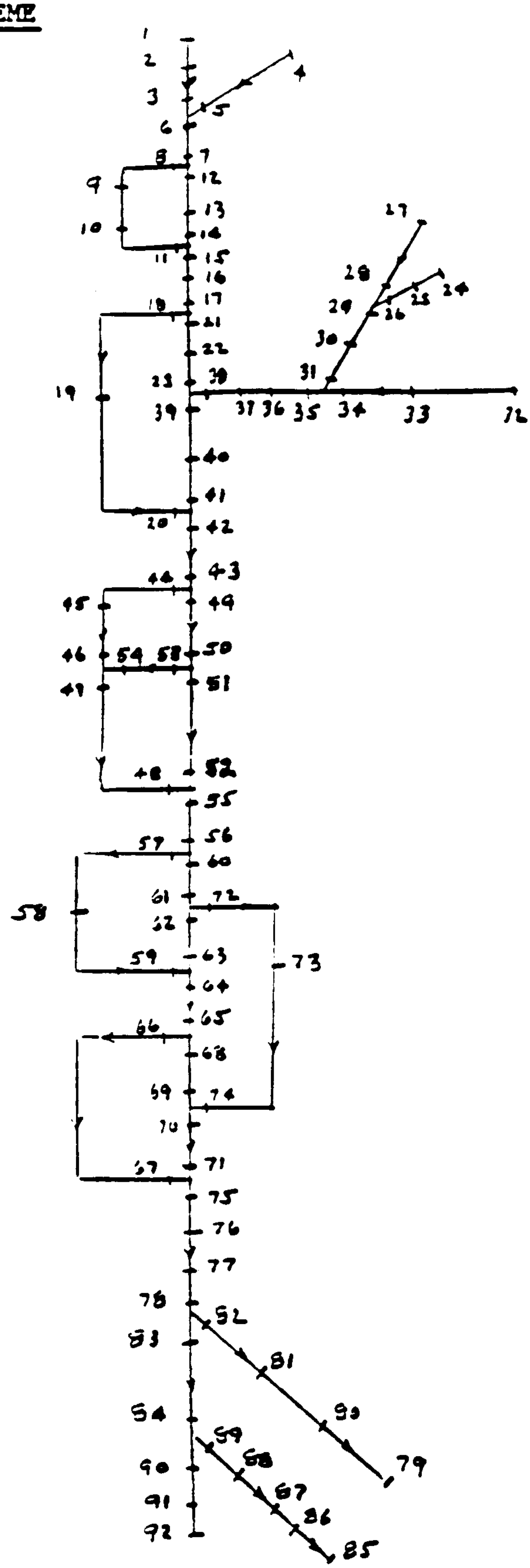
11 Critical Flow Internal Boundary

NODE NUMBER SCHEME

Example No. 2:

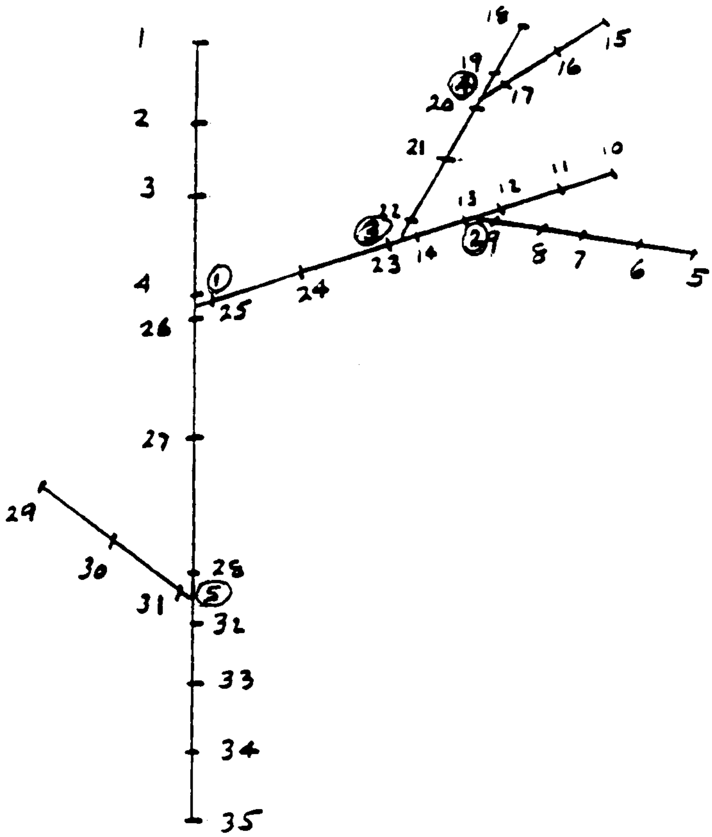
Junct. No.	Up. Node	Trib. Node	Down. Node
1	3	5	6
2	7	8	12
3	14	11	15
4	17	18	21
5	41	20	42
6	23	38	39
7	26	28	29
8	31	34	35
9	43	44	49
* 10	46	54	47
* 11	52	48	55
* 12	50	53	51
13	56	57	60
14	63	59	64
* 15	61	72	62
* 16	65	66	68
* 17	71	67	75
18	69	74	70
19	78	82	83
20	84	89	90

* Corrected from previous release.



NODE NUMBERING SCHEME

Example No. 1:



Junction No.	Upstream Node	Tributary Node	Downstream Node
1	4	25	26
2	9	12	13
3	14	22	23
4	17	19	20
5	28	31	32

APPENDIX C

ADDITIONAL INPUT DATA DESCRIPTION FOR "STORM SEWER" OPTION IN DWOPER

Version: 1/3/86

Card Input no.		Data Description - Input Format
(1A)	K21,K22,K23,K24,K25,K26 - 6 I 5	
	K23	Maximum number of manholes.
	K24	Maximum number of inlet hydrographs other than upstream boundaries.
	K25	Maximum number of detention or pumping basins
	K26	Maximum number of pumps.
(5)	NCS,NCSS,NP,KTERM,KPL,KPL2,JNK,NPEND - 8I10	
	NPEND	Sequential number indicating the last value in the computed stage hydrograph that will be used in the statistics needed in the Automatic Calibration option to determine the Manning n. If left blank, all computed values will be used. If (-1) or (-2) is entered, the Storm Sewer option can be used. A value of (-2) indicates the storm sewers are circular pipes. A value of (-1) indicates the storm sewers are of arbitrary shape.
(51E)	NMH(J),NQPU(J),NDTB(J),NPM(J) - 4I10	
	NMH(J)	Number of manholes in storm sewer system.
	NQPU(J)	Number of inlet hydrographs in storm sewer system.
	NDTB(J)	Number of detention or pumping basins in system.
	NPM(J)	Number of pumps in system.

(51F) NXPI(K,J),DMH(K,J),FKV(K,J),PCWR(K,J),DELVO(K,J),PWELV(K,J) - I10,4F10.2

NXPI(K,J) Sequential number of node (cross-section) which is an entrance to a manhole; if a manhole has two channels entering, use the smallest number to denote this parameter.

DMH(K,J) Diameter (ft) or equivalent diameter of manhole.

FKV(K,J) Head loss coefficient for manhole; varies from about 0.10 to 0.35 (see ASCE J. Hydr., Aug 1984, p. 1150-1154, "Head Losses at Sewer Junction Manholes," by J. Marsalek).

PCWR(K,J) Discharge coefficient for weir flow out of the top of the manhole that is permanently lost from the storm sewer system; varies from 3.1 to 3.3; if the flow out of the top of the manhole returns thru the manhole to the sewer as the flow levels in the system recede, PCWR (K,J) represents the length (ft) of a square area equivalent to the surface area that temporarily stores the water exiting thru the top of the manhole. If PCWR(K,J) is given a value of 10.0, the card (51G) is entered for the surface area.

DELVO(K,J) Elevation interval (ft msl) in table of surface area SAMH(L,K,J) and (51G) vs. elevation.

PWELV(K,J) Elevation (ft) of top of manhole.

(51G) SAMH(L,K,J)

SAMH(L,K,J) Surface area (acres) associated with a manhole where water is temporarily stored as it exits through the top of the manhole; L index goes from 1 to 3; surface area begins at elevation of top of manhole (PWELV) and increments at DELVO(H,J) intervals upward.

<u>Note</u>: (51G) is omitted unless PCWR(K,J) = 10.0.

<u>Note</u>: Repeat card 51 F once for each K, K=1, NMH(J). Omit card 51F if NMH(J) = 0.

(51H) NXQPU(K,J) - I10

NXQPU(K,J) Sequential number of node (cross-section) where an inlet hydrograph occurs.

(51I) QPU(I,K,J) - 8F10.2

QPU(I,K,J) Inflow (cfs) hydrograph ordinate value for K^{th} inlet hydrograph; I index goes from 1 to NU (See card 4).

Note: Repeat cards 51H and 51I once for each K, K=1, NQPU(J). Omit cards 51H and 51I if NQPU(J) = 0.

(51J) NXDTB(K,J),PEO(K,J),DBCW(K,J),DBWELV(K,J),DBCO(K,J) - I10,4F10.2.

NXDTB(K,J) Sequential number of node of inlet to manhole where a connection to a detention or pumping basin is located.

PEO(K,J) Elevation (ft msl) of water surface in detention or pumping basin at t=0.

DBCW(k,J) Weir discharge coefficient for inlet to detention or pumping basin; DBCW(K,J) = (3.1 to 3.3) x length of weir (ft).

DBWELV(K,J) Elevation (ft msl) of weir crest.

DBCO(K,J) Orifice discharge coefficient for inlet to detention or pumping basin; DBCO(K,J) = (0.51 to 0.98) x 8.02 x area of orifice (ft^2).

DBOELV(N,J) Elevation (ft msl) of top of orifice.

(51K) PSA(I,K,J) - 8F10.2

PSA(I,K,J) Volume (ft^3) of K^{th} detention or pumping basin for a specified elevation within the basin; I index goes from 1 to a minimum value of 2 or a maximum value of 8.

(51L) PEL(I,K,J) - 8F10.2

PEL(I,K,J) Elevation (ft msl) associated with each corresponding value of PSA(I,K,J); elevations start at the bottom of the basin and proceed upwards; I index goes from 1 to a minimum value of 2 or a maximum value of 8.

Note: Repeat cards 51J, 51K, 51L once for each K, K=1, NDTB(J).
Omit cards 51J, 51K, 51L if NBTB(J) = 0.

(51M) NXPO(K,J),NQLP(K,J),PEMN(K,J),PEMX(K,J),PELV(K,J) - 2I10,3F10.2.

NXPO(K,J) Sequential number of node of inlet to manhole where a connection to a basin with pumps is located.

NQLP(K,J) Parameter indicating if pump has a specified discharge hydrograph (NQLP(K,J) =1) or operates according to a specified head-discharge relation (NQLP(K,J) = 0)

PEMN(K,J) Elevation (ft msl) of water surface in pumping basin at which pump stops operating.

PEMX(K,J) Elevation (ft msl) of water surface in pumping basin at which pump starts operating.

PELV(K,J) Elevation (ft msl) at which pump discharges + approximate head loss thru discharge pipe.

(51N) DHP(I,K,J) - 8F10.2

DHP(I,K,J) Head differential (ft) in head-discharge relation; I index goes from 1 to minimum value of 2 or maximum value of 8; head begins at zero and increases.

(51P) OP(I,K,J) - 8F10.2

OP(I,K,J) Pump discharge (cfs) associated with each corresponding DHP(I,K,J) value.

Note: Omit cards 51M and 51N if NQLP(K,J) = 1.

(51Q) QLP(I,K,J) - 8F10.2

QLP(I,K,J) Pump discharge (cfs) hydrograph; or elevation (ft msl) at which pump discharges + approximate heat loss through discharge pipe; if PELV(K,J)=1.0, QLP(I,K,J) is a time-dependent value of PELV(K,J), I index goes from 1 to NU.

Note: Omit card 51Q if NQLP(K,J) = 0 and PELV(K,J) ≠ 1.0.

Note: Repeat cards 51M, 51N and 51P, or 51Q once for each K, K=1, NPM(J).
Omit cards 51M, 51N and 51P, or 51Q if NPM(J) = 0.

Note: Repeat cards 51E thru 51Q once for each J, J=1, JN where JN (card 4) is set to a value of 1 when NET (card 4) is either 1 or 2.

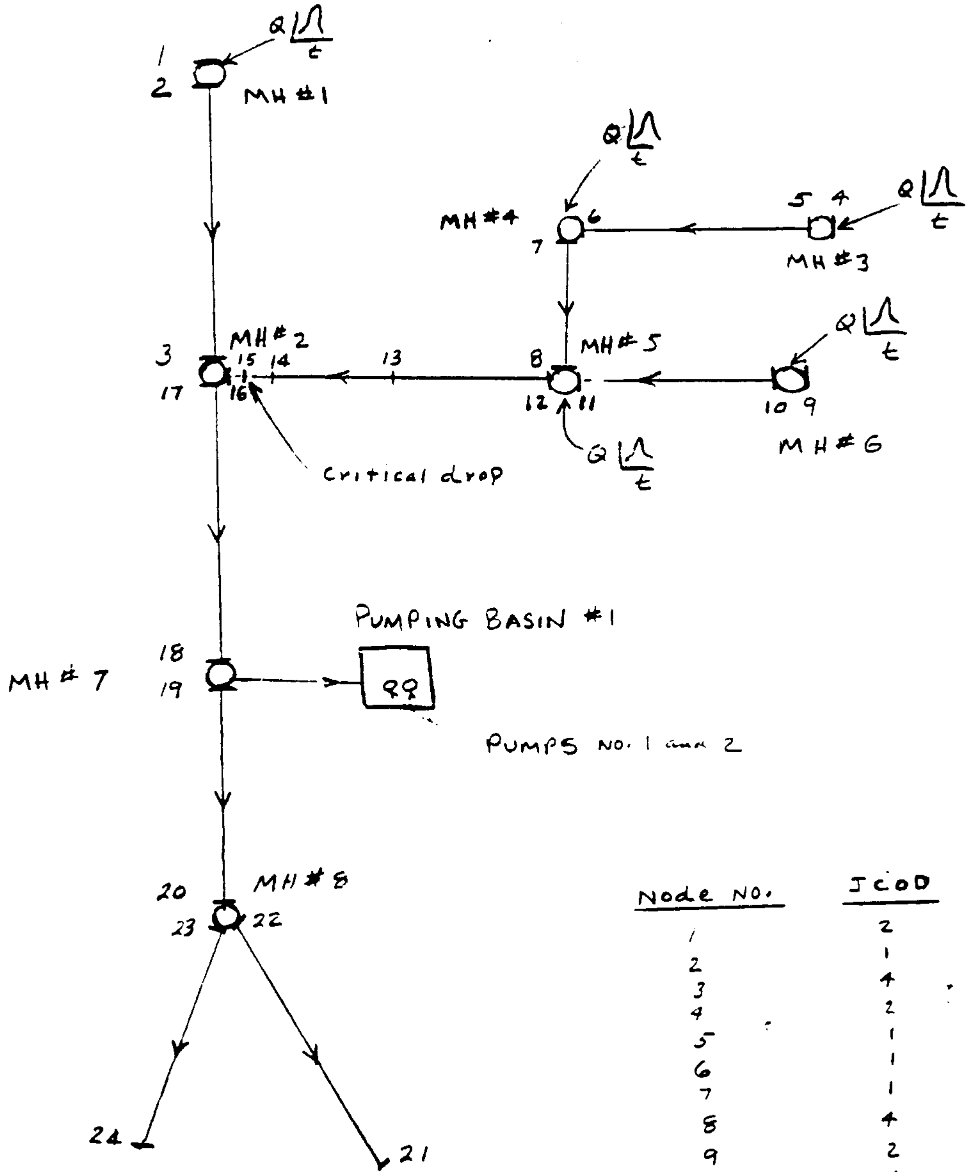
Note: Omit cards 51E thru 51Q if NPEND ≥ 0.

Note: When the storm sewer option is used and the sewers are circular pipes, only two values are required for the BS(K,I,J) top widths on card 28. The first value is the pipe diameter (ft) and the second value represents the diameter or width (ft) of the fictitious chimney emanating from the top of the sewer pipe. The K index goes from 1 to 2. Thus, the value of NCS (card 5) should be entered as a value of 2 when circular pipes comprise the storm sewer system.

Only one value is required for the HS(K,I,J) elevations (card 29). This value represents the invert (bottom) elevation (ft) of the pipe. The K index goes from 1 to 1.

The AS(1,I,J) values on card 30 are entered as zero's for either circular pipes or arbitrary-shaped sewers. Arbitrary-shaped sewers are described by a sufficient number of BS(K,I,J) values and corresponding HS(K,I,J) elevations.

NODE NUMBERING SCHEME



Junction No.	JU	JB	JL
1	3	16	17
2	8	11	12
3	20	22	23

Node No.	JCOD
1	2
2	1
3	4
4	2
5	1
6	1
7	1
8	4
9	2
10	1
11	4
12	4
13	1
14	1
15	11
16	4
17	4
18	1
19	1
20	4
21	2
22	4
23	4
24	3